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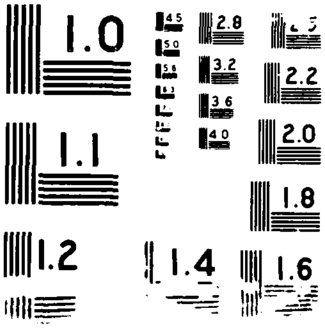
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Development of Design Formulas for Ribbed Mat Foundations on Expansive Soils

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Background information and procedures for the analysis and structural design of ribbed mat foundations on expansive soils are the basis for this technical report. Within the text are the basic rules and requirements for the design according to soil behavior. Different design methods are compared and the design formulas and construction details are presented with an objective for simplicity, accuracy, rationality, and flexibility.

The Tulsa District, Structural Section, conducted computer parametric studies on which a new design procedure was based and developed by the Southwestern Division, Structural Section.

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PREFACE

This report contains background information and procedures for the analysis and structural design of ribbed mat foundations on expansive soil. The new design procedure, developed by the US Army Engineer Division, Southwestern, Structural Section, is based on computer parametric studies conducted by the US Army District, Tulsa, Structural Section.

Work was coordinated through an advisory group consisting of Joseph Hartman, SWDED-TS, Jack Fletcher, SWDED-G, Garland Young, SWFED-DT, Al Branch, SWFED-FD, George Henson, SWTED-DT, Carl (Sandy) Stephens, SWTED-DT, Harrison Sutcliffe, SWTED-DT, George Hall, SWTED-GP, and Cliff Warren, SWTED-GP. Messrs. Hartman and Bill James, SWDED-TS, prepared this report. Funding was provided through Tulsa and Fort Worth Districts, Southwestern Division, and the Computer-Aided Structural Engineering (CASE) task group on Building Systems. Mr. Paul K. Senter, Acting Chief of the Information Research Division, Information Technology Laboratory (ITL) and Mr. Chris A. Merrill, Engineering Applications Office (EAO), reviewed and provided technical assistance for publication of this report at the US Army Engineer Waterways Experiment Station (WES). Ms. Gilda Miller, Editor, Information Products Division, ITL, WES, provided final editing of the material for this report before publication.

COL Dwayne G. Lee, CE, is the Commander and Director of WES. Dr. Robert W. Whalin is Technical Director.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
inches	25.4	millimetres
inches (force) per pound	0.1129848	metre-newtons
kips (force) per foot	1355.818	newton metres
kips (force) per square foot	47.88026	pascals
pounds (force)	4.448222	newtons
pounds (force) per foot	14.5939	newtons per metre
pounds (force) per inch	175.1268	newtons per metre
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	0.006894757	megapascals
pounds (mass) per cubic inch	27.6799	grams per cubic centimetre

DEVELOPMENT OF DESIGN FORMULAS FOR RIBBED MAT FOUNDATIONS ON EXPANSIVE SOILS

PART I: GENERAL REQUIREMENTS FOR RIBBED MATS

Background

1. Ribbed mat foundations consist of a thin slab on grade which acts monolithically with a grid of stiffening beams beneath the slab. The beams (ribs) are cast in trenches dug in the foundation soil. Ribbed mats combine the economic advantages of shallow foundations with the performance advantages of monolithic floors. Ribbed mats are especially useful for minimizing differential foundation movements in areas with expansive soils.

Expansive Soils

Behavior

2. Center lift. In the center-lift condition the soil near the edge of the slab drops in relation to the soil near the center. This is due to moisture retention by the interior soils and the drying and shrinking of perimeter soils. As this occurs, the perimeter soil provides less support for the edge of the slab which then acts as a cantilever. This is illustrated in Figure 1.

3. Edge lift. In the edge-lift condition the soil near the edge of the slab rises in relation to the soil near the center. This is due to the increasing moisture content and subsequent swelling of soil near the edge. The swelling soil raises the edge of the slab, causing some of the slab to lift off the soil. Interior loads cause the slab to sag and recontact the soil at some interior location. The slab thus tends to act as a beam, simply supported by the soil at the edge, and by soil support near the center of the slab. The amount of support at the center depends on numerous parameters such as interior loads, rib bending stiffness, soil-swell pressures, and the magnitude of soil swelling. Typical edge-lift behavior is illustrated in Figure 2.

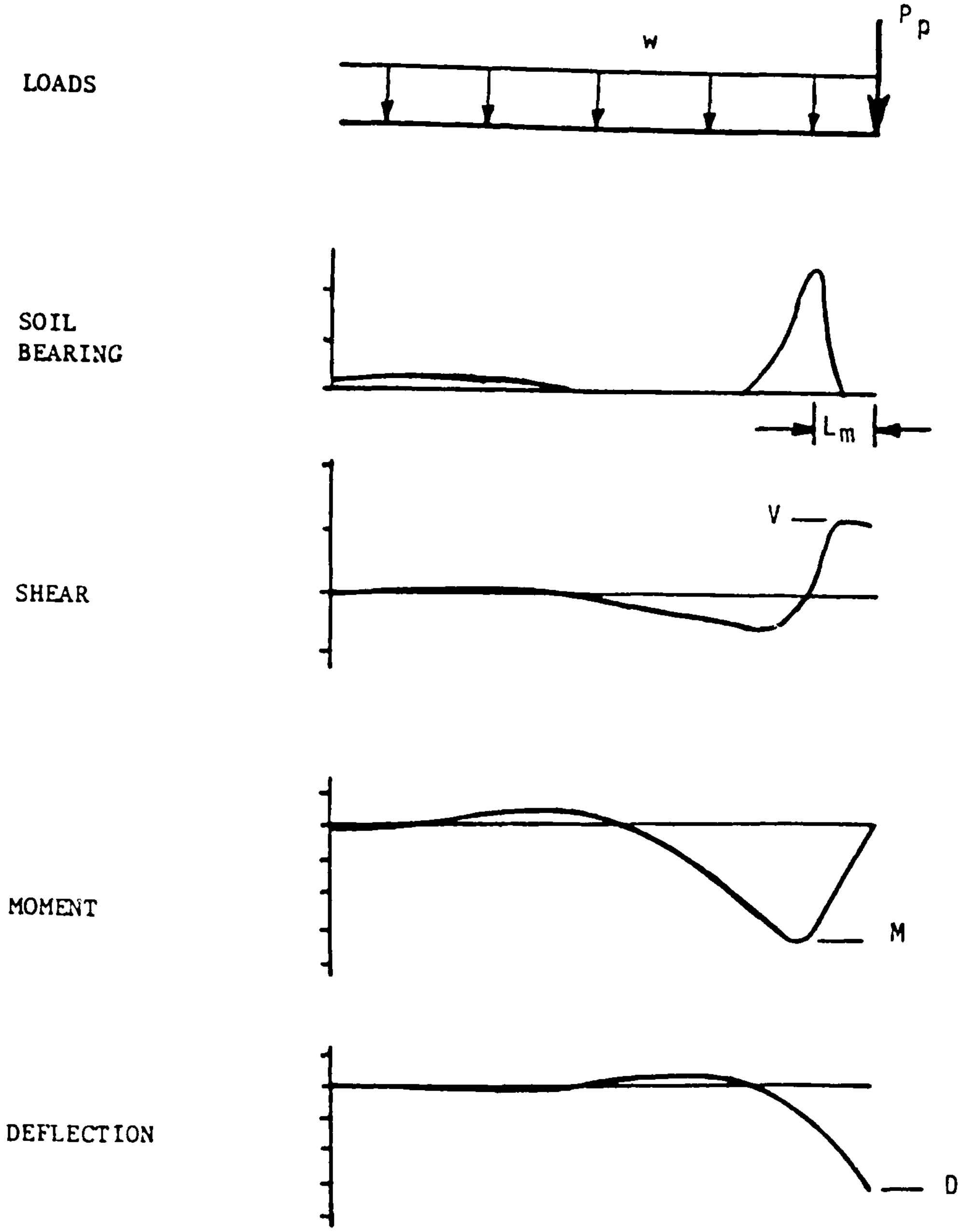

Figure 1. Center-lift behavior

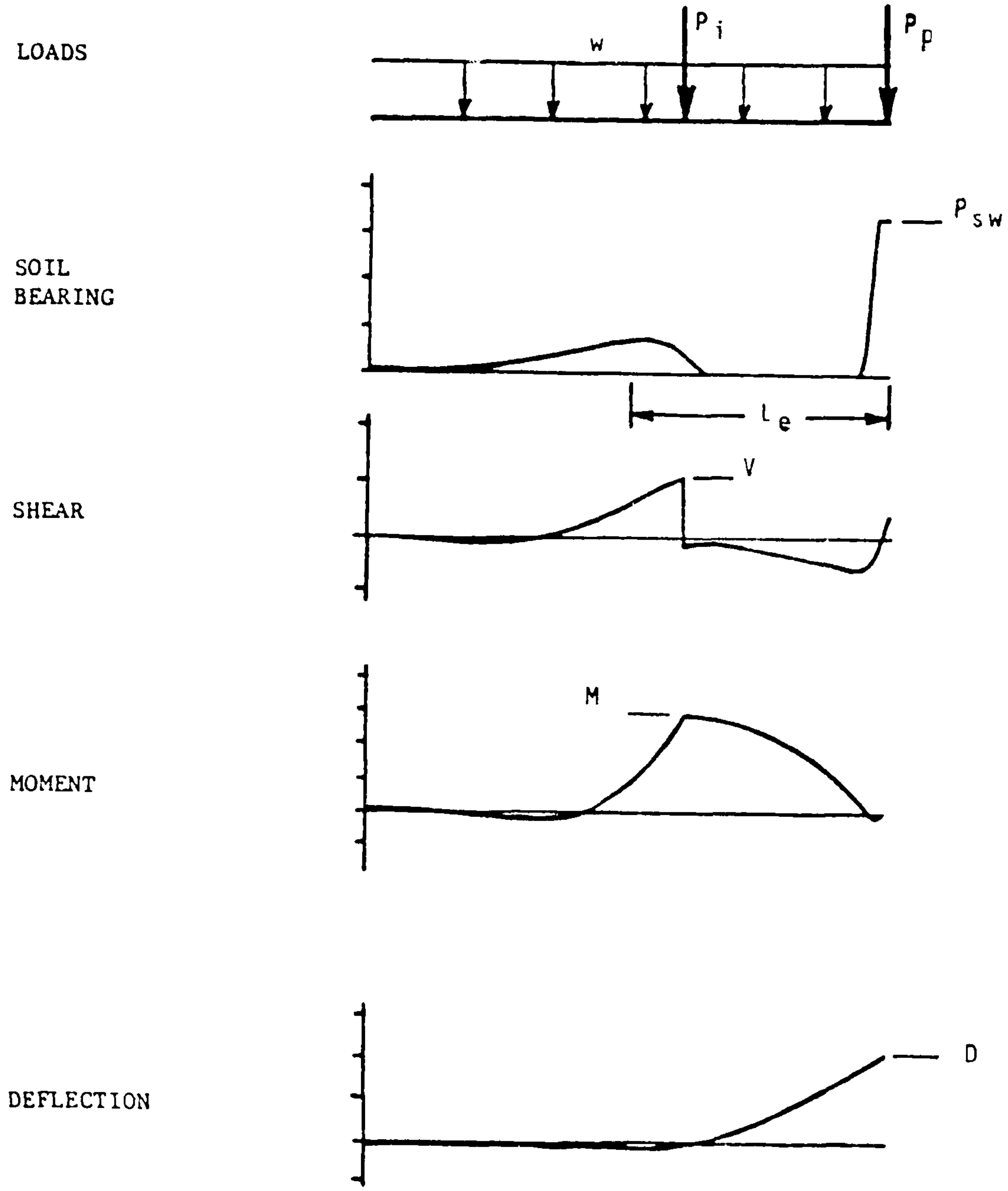

Figure 2. Edge-lift behavior

Design methods

4. Southwestern Division (SWD) method.* All ribbed mats on expansive soils shall be designed in accordance with the provisions of Part II of this report. However, ribbed mats for family housing may be designed in accordance with paragraphs 5 and 6.

5. Post Tensioning Institute (PTI) method.** The PTI method may be used only for design of family housing foundations on expansive soils. Specifically, slab width (short dimension) should not exceed 40 ft,† rib depths should not exceed 30 in., loading should consist only of perimeter loads and light interior distributed loads (DL + LL ≤ 100 psf), soils should be fairly weak in situ materials with no extensive substitution of nonexpansive fill. When using the PTI method, the following provisions shall apply: Rib spacing shall not exceed 15 feet; concrete tensile stress shall not exceed $4\sqrt{f'_c}$; the minimum effective prestress shall be 100 psi.

6. Building Research Advisory Board (BRAB) method.†† The BRAB report may be used only for design of foundations for family housing. However, the PTI method is preferred, since the BRAB method may produce unreasonable results for large foundations.

7. Computer method. In lieu of paragraph 4, ribbed mats may be designed using appropriate computer programs. Such programs must be capable of modeling the variable soil swell due to moisture changes, and the nonlinear soil-structure interaction near the perimeter of the foundation. One such computer program is CBEAMC.‡

8. Load factors. When using the above methods to design ribbed mats for center-lift and edge-lift conditions, load factors may be multiplied by 0.75 (strength method) or allowable stresses may be increased by one third (working stress method). This provision does not apply to the allowables

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* US Army Engineer Division, Southwestern. Engineering Instruction Manual, current edition.

** Post Tensioning Institute. 1980. "Design and Construction of Post-Tensional Slabs-on-Ground," 1st ed., Phoenix, Ariz.

† A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

†† Building Research Advisory Board. 1968. "Criteria for Selection and Design of Residential Slabs-on-Ground," prepared for Dept. of Housing and Urban Development, National Academy of Sciences, Washington, DC.

‡ US Army Engineer Waterways Experiment Station. 1982 (Jun). "User's Guide: Computer Program for Analysis of Beam-Column Structures with Nonlinear Supports (CBEAMC)," Instruction Report K-82-6, Vicksburg, Miss.

given for the PTI method, since those allowables have already been increased from the usual provisions of ACI 318-83.*

Nonexpansive Soils

9. Ribbed mat slabs on nonexpansive soils need not be designed for bending due to center-lift or edge-lift conditions. Beam on elastic foundation analyses may be used to determine the effects of concentrated loads on ribs, or ribs may be designed as conventional strip or spot footings.

Soil Properties

10. Soil properties for design of ribbed mats will be as provided in the "Foundation Design Analysis" by the Corps of Engineers.** Criteria for developing these properties is included in SWD criteria Letter XV 7-12.† Properties necessary for design in accordance with paragraph 4 consist of the following, which are defined in Appendix A:

q_a - allowable bearing pressure
k - subgrade modulus
Y_m - soil heave
L_m - edge moisture variation distance
P_{sw} - pressure of swelling soil acting on perimeter rib

Minimum Requirements

Subgrade preparation

11. A vapor barrier, capillary water barrier, and a minimum of 18 in. of nonexpansive fill will normally be used beneath ribbed mats. Additional nonexpansive fill will often be used to lessen the effects of highly expansive soils. These requirements will be detailed in the "Foundation Design Analysis" (unpublished site-dependent report footnoted on this page).

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* American Concrete Institute. 1983. "Building Code Requirements for Reinforced Concrete (ACI 318-83), ACI Committee, Detroit, Mich.

** US Army Engineer District, Vicksburg, Geotechnical Branch. "Foundation Design Analysis," a site-dependent report (unpublished).

† Letter, SWDED-G, 16 April 1987. "Criteria for Developing Geotechnical Design Parameters for Ribbed Mat Design Methodology."

Slab

12. For family housing and other small lightly loaded buildings, a 4-in. slab may be used. For other buildings, the minimum slab thickness will be 5 in. Minimum slab reinforcing shall be 0.2 percent. Where slabs are subjected to vehicular loading, they must be designed for the maximum wheel load, similar to paving. Use 650-psi flexural strength concrete for slabs subject to wheel loads.

Grid geometry

13. Ribs should be located to form a continuous grid. Rib spacing should not exceed 20 ft in expansive soils, or 25 ft in nonexpansive soils. Locations of ribs should conform to significant wall and column loads, and may be used to resist thrusts from rigid frame reactions. Ribs should be provided around large openings in the slab. In expansive soils, diagonal ribs are required at exterior corners. Expansion joints should be provided at 250-ft intervals, and should also be used to break irregularly shaped buildings into rectangular segments. Foundations for family housing do not require expansion joints due to irregular shapes.

Rib size

14. Minimum rib depth is 20 in. Rib depths should usually not exceed 3 ft to minimize construction difficulties related to placing reinforcement and maintaining trench walls. If deeper ribs are used, rib width should also be increased. Minimum rib width is 12 in. except for family housing foundations where 10-in. ribs may be used. Sufficient rib width must also be provided to transfer wall and column loads to the soil as strip footings. The allowable soil bearing capacity may not be exceeded when considering the width of the rib plus an effective slab width on each side of the rib. The effective slab width for bearing is limited to the thickness of the slab. At column locations an alternate is to provide fillets at rib intersections, sufficient to act as spot footings for column loads.

Rib capacity

15. Concrete should have a minimum compressive strength of f'_c = 3,000 psi at 28 days. Reinforcing shall be grade 60, except ties may be grade 40. Minimum reinforcing ratio (A_s/A_g) shall be 0.0033 top and 0.0033 bottom, and this may be reduced to 0.005 total in nonexpansive soils. Use No. 3 ties at 24 in. minimum. These minimums should be sufficient for shrinkage stresses and for unpredictable soil behavior.

Prestressed mats

16. For prestressed ribbed mats, not designed per PTI, all the minimum requirements apply except that slab and rib top reinforcement may be deleted and replaced by appropriate posttensioning strands. Mild steel shall still be provided in the bottom of ribs. Minimum prestress shall be 100 psi on the gross area of the slab, including effects of subgrade friction as calculated by the PTI method. Concrete tensile stress shall be limited to $3\sqrt{f'_c}$ and shear stress limited to $1.1\sqrt{f'_c}$. A one-third overstress may be allowed per paragraph 8.

Construction Details

Conventionally reinforced

17. Construction joint spacing should not exceed 50 ft in either direction. A horizontal construction joint may be provided in the ribs at the base of the capillary water barrier when unstable trench walls may cause construction difficulties. However, this is discouraged because of increased potential for shrinkage cracks in the slab.

Prestressed

18. Construction joint spacing shall not exceed 75 ft in either direction. Tendons within each placement shall be stressed to 15 percent of the final prestress not more than 24 hours after the concrete has attained sufficient strength to withstand the partial prestress. Other construction procedures for prestressed ribbed mats shall conform to the PTI method.

Contractor designs

19. Ribbed mat foundations may be designed as prestressed or conventionally reinforced as selected by the engineer. The plans and specifications shall not include the option of changing the ribbed mat from one type to another. The reason for this prohibition is that design parameters (e.g., moments of inertia) may be dependent on the type of ribbed mat being designed and may affect calculated shears and moments. This does not prohibit revisions of the slab type as a result of contractor value engineering proposals. However, such revisions must include a complete design of the ribbed mat foundation using appropriate design parameters in accordance with this report.

PART II: ANALYSIS OF RIBBED MAT FOUNDATIONS ON EXPANSIVE SOILS

Scope

20. This part of the report contains the basic rules for design of ribbed mats in expansive soils. This method may be used to predict shears, moments, and deflections in ribs subject to soil movement due to changing moisture content. For a commentary on the design method refer to Part III; for example design calculations refer to Appendix A. The design method from Part II should be used in conjunction with the "minimum requirements" for ribbed mats, as presented in Part I.

21. The Notation is presented for clarity and convenience in reading this report:

C = Correction factor for equivalent cantilever length

D = Beam deflection (in.)

I = Moment of inertia per foot, $I = I_r/S$ (in.^{4}/ft)

I_r = Moment of inertia of rib (in.^{4})

k = Modulus of subgrade reaction (pci)

L_o = Basic length of cantilever (ft)

L_c = Equivalent length of cantilever, center lift (ft)

L_e = Equivalent length of simple beam, edge lift (ft)

L_i = Distance from perimeter to location of interior load (ft)

L_m = Edge moisture variation distance (ft)

L_b = Width of soil bearing at perimeter, edge lift (ft)

M = Bending moment per foot (ft-kip/ft)

M_r = Bending moment per rib, $M_r = M_x S$ (ft-lb)

P_i = Interior load (plf)

P_p = Perimeter load (plf)

P_{sw} = Pressure of swelling soil on perimeter rib (psf)

R = End reaction at perimeter for equivalent simple beam (lb)

S = Rib spacing (ft)

w = Uniform load (psf)

V = Shear per foot (lb/ft)

V_r = Shear per rib, $V_r = V_x S$ (lb)

Y_m = Soil heave (in.)

θ = Rotation of support of equivalent cantilever (rad)

Units

22. The equations presented in paragraphs 33 through 35 are written for units as defined in the Notation. If other units are used, the equations must be modified appropriately.

Rib definitions

23. Ribs are defined as perimeter, transverse, or diagonal as shown in Figure 3. Note that transverse refers to ribs parallel to either axis of the building.

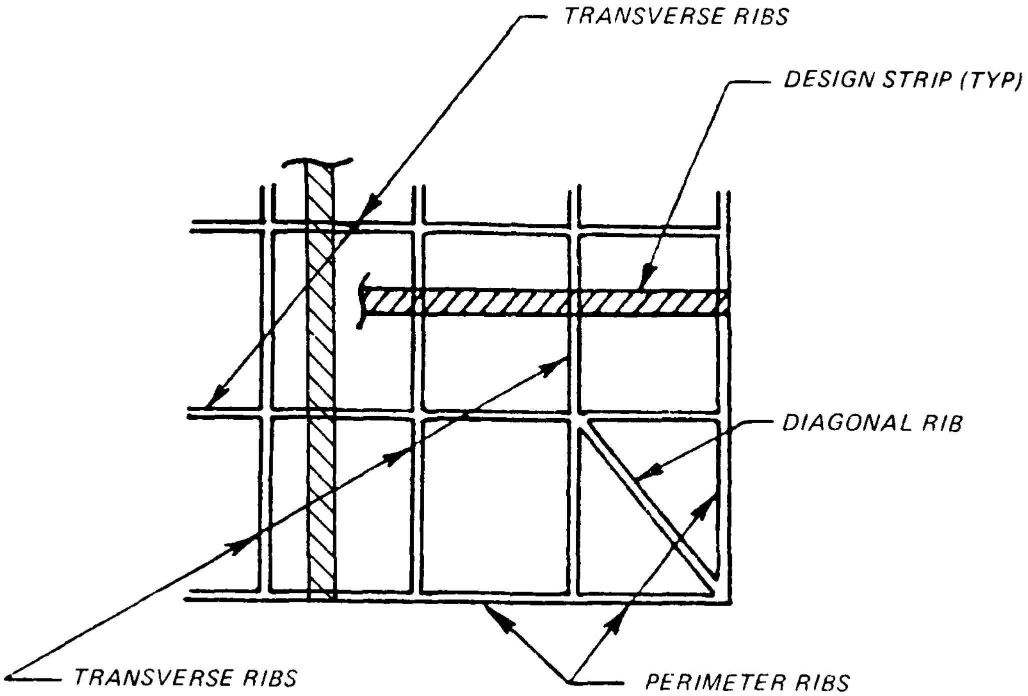


Figure 3. Rib definitions

Strip analysis

24. The analysis is based on a strip assumption, ignoring the effects of the grid configuration of the ribs. The formulas and examples presented below are for an equivalent 1-ft strip, using "per foot" values for loads and stiffness.

Soil-edge profile

25. For edge lift the maximum swell occurs at the perimeter and decreases rapidly toward the interior. The soil profile is assumed to be parabolic (in the unloaded condition) and is illustrated in Figure 4.

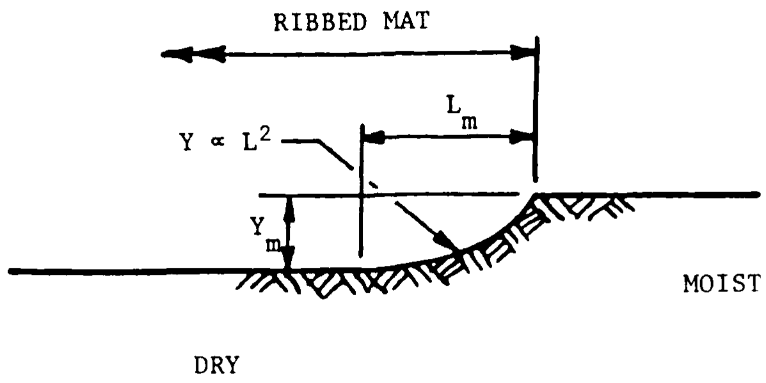


Figure 4. Soil-edge profile

Analysis Method

Transverse rib - Center lift

26. Center-lift analysis is based on an equivalent cantilever beam to determine moments, shears, and deflections.

27. Moment. The length of the equivalent cantilever can be calculated as:

$$L_c = C \times L_o$$

where

$$L_o = 2.3 + 0.4\, L_m$$

$$C = \frac{0.8 \times Y_m^{0.12} \times I^{0.16}}{P_p^{0.12}}$$

The maximum moment may then be calculated from statics using conventional cantilever formulas such as:

$$M = P_p L_c + \frac{1}{2}\, w\, L_c^2$$

The moment can then be assumed to be constant for a distance $L_c/2$ and then to decrease linearly from M at the cantilever support, to near zero at a

distance $5L_c$ from the perimeter. To obtain the design moment for a given rib, multiply the calculated per-foot moment by the appropriate rib spacing ($M_r = M \times S$).

28. Shear. The maximum shear may be calculated from statics using the same equivalent cantilever as for moment.

$$V = P_p + w\,L_c$$

The shear may then be assumed to decrease linearly from V at the cantilever support, to near zero at a distance $5L_c$ from the perimeter. To obtain the design shear for a given rib, multiply the calculated per-foot shears by the appropriate rib spacing ($V_r = V \times S$).

29. Deflection. Deflection at the perimeter is the sum of three components: bending deflection of the equivalent cantilever, vertical translation of the cantilever support, and rotation of the cantilever support. Rotation of the support may be calculated as:

$$\theta = \frac{M^{1.4}}{9{,}800\ I\ k^{0.5}}$$

The perimeter deflection is then:

$$D = 0.11 + 12\,L_c\,\theta$$

where 0.11 in. is an approximation for the support translation plus the cantilever bending, and ($12\,L_c$) is the length in inches.

30. Use the deflection calculated above to compare with allowable deflection. The allowable deflection may be determined by using $4L_c$ as the length between points of zero and maximum deflection.

Transverse rib - Edge lift

31. Edge-lift analysis is based on an equivalent simple beam, supported at the perimeter and at some interior location.

32. Deflection. The first step in calculating deflection is to determine the length of the equivalent simple beam. The appropriate length depends on many parameters, including the deflection. Therefore, deflection must

first be estimated to determine equivalent length, then a deflection is calculated based on that length. The process is repeated until calculated deflection matches the assumed deflection. The equivalent simple beam length may be calculated as:

$$L_e = \frac{7.5\ I^{0.17}\ L_i^{0.37}\ D^{0.12}}{W^{0.07}\ P_i^{0.11}}$$

The perimeter end reaction (R) for this beam may be calculated from statics. For an ideal case the reaction is:

$$R = P_p + \frac{1}{2}\, w\, L_e + \left(\frac{P_i (L_e - L_i)}{L_e}\right)$$

The width of soil bearing at the perimeter can be approximated as:

$$L_b = 1.1\left(\frac{R}{P_{sw}}\right)$$

where P_{sw} is selected from a curve of heave versus bearing pressure, corresponding to the estimated deflection used during this iteration. The edge deflection is found by determining the soil swell at a distance L_b from the perimeter, based on the parabolic swell profile:

$$D = \frac{Y_m \left(L_m - L_b\right)^2}{L_m^2}$$

When satisfying deflection criteria, use the calculated deflection and equivalent simple beam length.

33. Moment. Once the simple beam equivalent length has been determined, the bending moments may be calculated based on statics. To obtain rib design moments, multiply per-foot moments by the rib spacing.

34. Shear. Once the simple beam equivalent length has been determined, the shears may be calculated based on statics. To obtain rib design shears,

multiply per-foot shears by the rib spacing. Near the interior support the design shear need not exceed:

$$V = P_i + w(L_e - L_i)$$

This is due to the effects of distributed soil support, rather than the point support assumed in the simple beam analysis.

35. Special cases. If $P_i = 0$ or if $L_i > L_e$, make the following substitution in the above equation for L_e

$$1.4 = \frac{L_i^{0.37}}{P_i^{0.11}}$$

The equation for the simple beam length then becomes:

$$L_e = \frac{10.5\ I^{0.17}\ D^{0.12}}{w^{0.07}}$$

Perimeter rib

36. Center lift. For center lift the perimeter rib will have no support from the soil and must be designed to span between transverse ribs for the perimeter wall loads.

37. Edge lift. For edge lift the soil pressure on the perimeter rib will exceed the applied perimeter loads. The perimeter rib must be designed to span between transverse ribs for this net upward force.

Diagonal rib

38. Diagonal ribs are used to support exterior corners for center lift conditions, if loss of support occurs under both perimeter ribs. Diagonal ribs must be designed to provide the same moment and shear capacity as the larger of the two adjacent transverse ribs.

Interior rib

39. Interior ribs and rib intersections should be located at significant wall and column loads. The ribs should be designed for these loads as strip or spot footings, using beam-on-elastic-foundation methods. Differential soil movement due to moisture change is assumed not to occur except at the perimeter. However, to account for unpredictable interior soil movements, interior ribs must have the minimum size and capacity as required in Part I.

PART III: COMMENTARY

Hand Solutions Versus Computer Results

40. Actual behavior of ribbed mats in expansive soils involves complex, nonlinear, soil-structure interaction. The best solution for such behavior is provided by computer programs. The hand design method has been developed to approximate such computer results. Hand solutions have been checked by computer analyses; results have been within acceptable limits of error. However, such checks have been made only for a limited range for each design parameter, as shown in Table 1, corresponding to the usual values for military construction within SWD. If a wider range of parameters is applied to the hand design formulas, the results may be less accurate.

Notation

41. For nonprestressed rib mats the moment of inertia of a rib (I_r) should be the effective moment of inertia, calculated per ACI 318, Section 9.5.2.3.

42. The modulus of subgrade reaction (k) is the ratio of the soil pressure at the base of the concrete and the corresponding settlement. Since modulus values are typically determined by a plate-load test at the ground surface, they should be corrected for depth and for footing size (expected high pressure area between concrete and soil). Analyses have indicated that the high bearing pressure area for center-lift conditions will occur in an area several feet long parallel to the transverse rib and several feet on each side of the rib. A crude approximation for this area would be 5 ft square. This approximation should be adequate for design since calculations are not sensitive to the modulus of subgrade reaction.

43. The allowable bearing pressure (q_a) is the safe bearing capacity of the soil at the base of the ribs. A factor of safety of 3.0 is recommended for computing this value.

44. The edge-moisture variation distance (L_m) represents the distance, inward from the edge of the slab, over which the moisture content of the soil changes. Much judgement is required in determining this value.

45. The pressure of swelling soil on the perimeter ribs (P_{sw}) is the interface pressure between the soil and the base of the exterior rib, due to

an increase in soil-moisture content. The pressure which can be exerted by the swelling soil is dependent on the amount the surface of the soil is allowed to rise. Therefore, P_{sw} is usually presented as a curve of pressure versus heave. The actual upward deflection of the edge of the slab is a complex interaction between swell potential, structural loads, and mat stiffness, all of which combine to determine the interface pressure near the perimeter.

46. Soil heave (Y_m) is the differential vertical movement of the soil representing soil heave (edge lift) or soil shrinkage (center lift). The magnitude of Y_m is the computed vertical movement of a particle of soil at the ground surface due to a change in moisture content. This value should be based on the accumulation of potential volume changes for the full thickness of the active zone (Z_a), with no significant loads applied to the foundation. The value of Y_m may differ for edge-lift and center-lift conditions.

47. The applied loads (P_i, P_p, w) should consist of full dead plus live loads; including dead load of the slab and ribs.

Strip analysis

48. The hand solution formulas have been developed for analysis of an equivalent 1-ft strip. This is convenient for uniform loads and for soil properties, but requires some calculations for appropriate concentrated loads and bending stiffness. Rib stiffness must be divided by rib spacing to get the per-foot stiffness. If column loads exist they must also be divided by the rib or column spacing to provide an equivalent load per foot. If interior wall loads are parallel to the transverse rib, they must be divided by the rib spacing. These calculations are illustrated in Appendix A.

Soil-edge profile

49. The edge-lift condition occurs when increased moisture content swells exterior soils, and this effect extends under the edge of the slab. The center-lift condition occurs when soils under the slab are generally moist and seasonal drying occurs on the exterior, again extending under the edge of the slab. This causes the soil at the edge to shrink away from the slab.

50. The analysis method is based on an assumed parabolic swell profile which occurs uniformly along the perimeter. This is a convenient idealization of real soil behavior, which must be more erratic. However, the parabolic profile has better correlation with measured swells than do other possible edge profile assumptions. Note that the soil profile is not used in the hand design formulas for center lift. However, a parabolic profile was used in the

computer analyses for center lift, which formed the basis for the hand design formulas.

Design Method

51. Many of the formulas for shears, moments, and reactions are idealized, assuming P_p and R are exactly at the perimeter and that w extends to the perimeter. These approximates should usually be acceptable, but the formulas may be modified to account for actual load patterns.

Transverse rib - Center lift

52. Typical behavior of a transverse rib for center-lift conditions is shown in Figure 1. This illustrates the soil-bearing pressure and the shear, moment, and deflection. Note that the effects of the soil movement extend much farther than the moisture variation distance. The moment and shear distribution close to the edge resemble cantilever behavior.

53. Moment. The extent of significant moments is illustrated in Figure 1. The length of the equivalent cantilever can be taken as a basic length (L_o) which is dependent on the moisture variation distance, times a correction factor (C) which accounts for secondary effects of several parameters. The value of C will usually be slightly greater or less than unity. The C was developed to permit accurate approximations of computer results. It was developed from the ratios of actual values to usual values for significant parameters. For example, the "usual" values are: $Y_m = 1$ in. , $I = 1,500$ in.^{4}/ft , $P_p = 3,000$ lb/ft . Thus:

$$C = \left(\frac{Y_m}{1.0}\right)^{0.12} \left(\frac{I}{1,500}\right)^{0.16} \left(\frac{3,000}{P_p}\right)^{0.12}$$

$$C = \frac{0.8\ Y_m^{0.12}\ I^{0.16}}{P_p^{0.12}}$$

A similar approach was used to develop all the formulas in Part II which have an exponential format.

54. Shear. Maximum shear occurs near the support of the equivalent cantilever. The extent of significant shears is illustrated in Figure 1.

55. Deflection. Formulas for deflection include an assumed concrete modulus of elasticity $E_c = 3{,}320{,}000$ psi , for both center lift and edge lift.

56. Vertical movement at the perimeter is much greater than the bending deflection of the equivalent cantilever. To predict the deflection, it is necessary to consider translation and rotation at the support of the equivalent beam. The most significant component is due to rotation at the support. These components of deflection are shown in Figure 5. The sum of the cantilever bending and the support translation are approximated by the value 0.11 in. The percent error due to this approximation is negligible when total deflections are large. The percent error is greater when total deflections are small, but then the deflections are not significant anyway.

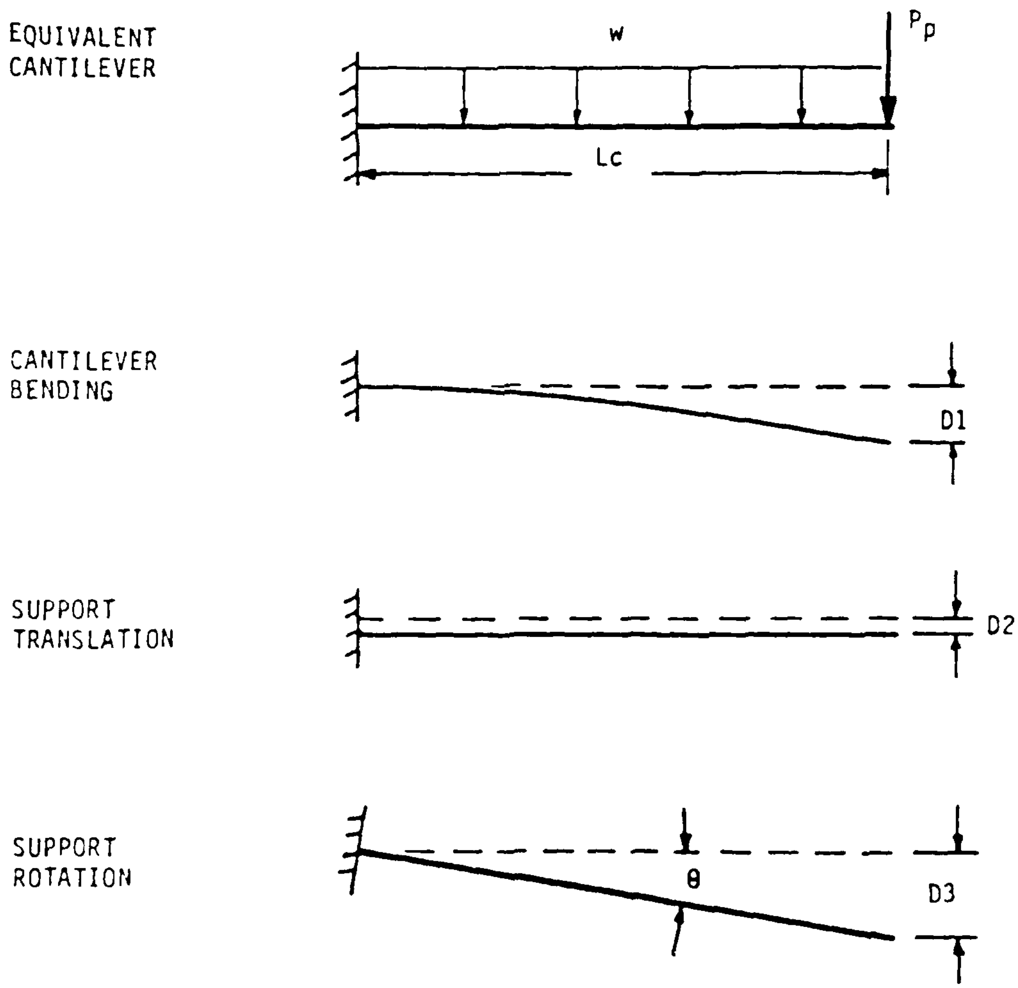
57. Allowable deflections* are expressed as a ratio of the difference in vertical movement at any two points compared to the distance between those points. For example: $D \le L/600$, where D is the differential displacement. In such formulas it is appropriate to use the point of maximum deflection and a point of near-zero deflection as the two measuring points. For center-lift behavior the maximum deflection occurs at the perimeter, and deflections tend to die out at approximately $4L_c$ (four times the equivalent cantilever length) from the perimeter. Therefore, the ratio $D/4L_c$ is appropriate for comparison with allowable deflections.

Transverse rib - Edge lift

58. Typical behavior of a transverse rib for edge-lift conditions is shown in Figure 2. This illustrates the soil bearing pressure and the shear, moment, and deflection. Soil swell lifts the edge of the ribbed mat, which actually rises off the soil for some distance from the perimeter. For shear and moment, this portion of the rib acts as a simply supported beam spanning between soil support at the perimeter and at an interior location.

59. Deflection. Vertical movement at the perimeter is driven by the tendency of the soil to swell, and is resisted by the downward loads applied on the soil. As the soil swells at the perimeter, the slab is lifted off the interior soil. This concentrates soil reactions near the edge, causing very high pressures. The pressures rise so high that they match the swell pressure of the soil. Thus, the soil cannot swell as much as it would if not loaded.

* US Army Engineer Division, Southwestern. Current edition. Engineering Instruction Manual.



$$D = D1 + D2 + D3$$

$$D1 + D2 = 0.11$$

$$D3 = 12\ Lc\ \theta$$

Figure 5. Center-lift deflection

Deflections can be predicted by balancing the upward force of the soil (the swell pressure times the bearing width) with the downward force of applied loads. This downward force can be determined from statics once an equivalent simple beam length is determined. The method for determining the deflection is shown in Figure 6.

60. Allowable deflections are expressed as ratios, as discussed in the commentary on paragraph 57. From Figure 2 it can be seen that the appropriate values for this ratio are the edge deflection and the equivalent simple beam length (D/L_c).

61. Edge-lift deflections are mainly a function of soil properties and applied loads, with bending stiffness of the ribs having only a secondary effect. Therefore, it may not be possible to control deflections by increasing the rib stiffness. It may be necessary to accommodate calculated deflections by using a less brittle superstructure or by detailing the superstructure to make it less sensitive to deflections. However, it may be necessary to modify soil properties to minimize the edge heave.

62. Moment. The moments can be calculated by statics, using the equivalent simple beam. The maximum moment will occur at the point of zero shear. Note that the maximum moment is quite sensitive to the beam length, therefore the iterative solution for deflection must converge accurately before calculating moments.

63. Shear. Shears can also be calculated by statics from the equivalent simple beam. Note that shears will reduce gradually to near zero around the interior end of the beam because of the distributed soil support.

64. Special cases. If no concentrated interior load exists or if it is very far from the perimeter, the formula for the simple beam length must be adjusted as shown. This adjusted formula was also developed to duplicate results from computer solutions.

Interior rib

65. Potential soil heaves in the interior are unpredictable and are generally due to localized moisture conditions, for example, due to a leaking pipe. Such conditions cannot be accounted for by design formulas. Adequate strength and stiffness for such unpredictable heaves should be supplied by the minimum requirements listed in Part I of the report. For interior wall or column loads the interior ribs should be designed in accordance with Part I, paragraph [illegible].

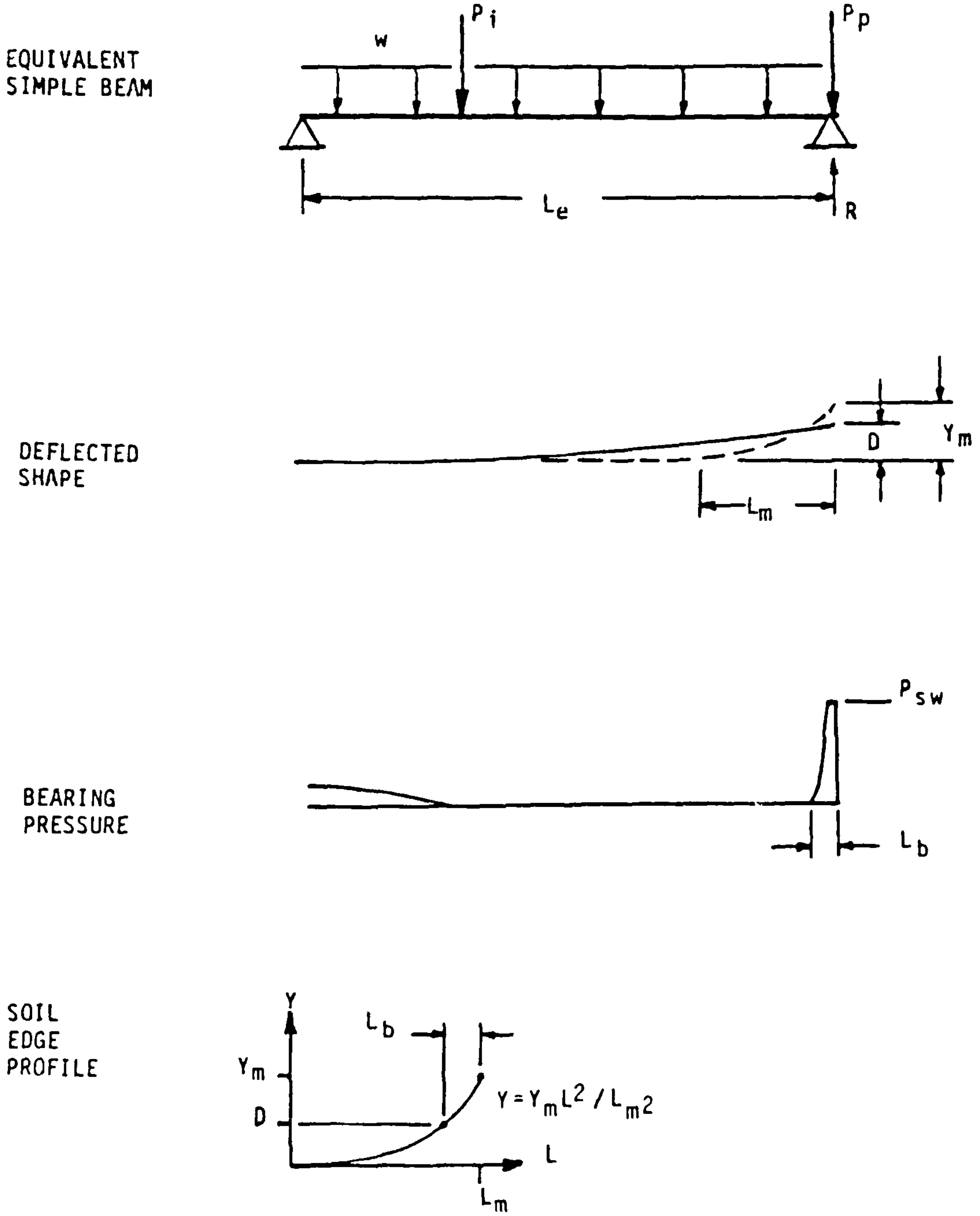


Figure 6. Edge-lift deflection

PART IV: THEORETICAL BASIS FOR PROCEDURE

Introduction

66. This part of the report contains background information which led to the development of design formulas presented in Part II. These formulas apply only to structural design of ribbed mat foundations on expansive soils. Previous design formulas were judged to be inadequate for general application within the US Army Engineer Division, Southwestern. The new formulas were developed to provide an adequate design method, other than performing a nonlinear soil-structure interaction analysis. Such computer analyses were used, however, to provide the basis for development of the new formulas. These analyses were performed by the US Army Engineer District, Tulsa, Structural Section, under the direction of the advisory group named in the Preface.

Computer Analysis

Computer program

67. The program used to analyze a ribbed mat foundation was CBEAMC.* This program was used to analyze a model consisting of a beam supported by nonlinear springs.

Computer model

68. Beam. The beam used in the computer model represented the smeared bending stiffness of a 1-ft strip of a typical ribbed mat. The beam extended from the perimeter, 30 ft towards the interior of the mat. Symmetrical boundary conditions were applied at the interior end. Such end conditions are appropriate since results indicate that perimeter soil behavior has little effect at that distance. Parameters used to describe beam stiffness included the effective rib moment of inertia (I_r) and the rib spacing (s). The smeared stiffness (I') was taken as $I' = I_r/s$. The effective moment of inertia may represent the bending stiffness of a tee beam formed by a rib plus an effective width of slab acting as a top flange.

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* US Army Engineer Waterways Experiment Station, 1982 (Jun), User's Guide: Computer Program for Analysis of Beam-Column Structure with Nonlinear Supports (CBEAMC), Instruction Report K-82-6.

69. Soil. Soil support for the mat was represented by nonlinear Winkler springs. Stiffness of the springs for downward displacement was dependent on the assumed subgrade modulus (k); upward displacement would result in loss of contact between mat and soil. The basic spring behavior is shown in Figure 7. Near the exterior end of the beam, soils would be subject to

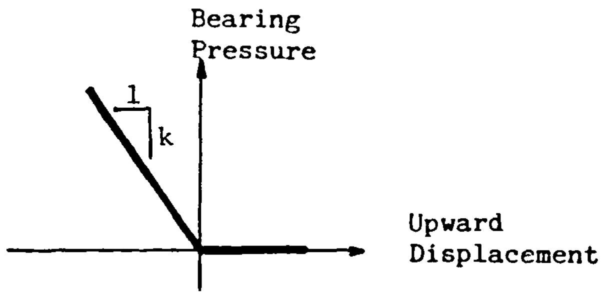


Figure 7. Basic soil spring

moisture-induced volume changes. Soil shrinkage would result in loss of support near the perimeter, a condition referred to as center lift. Soil swell would result in lifting of the perimeter of the mat, a condition referred to as edge lift. The extent of soil shrinkage or swell is defined by the edge-moisture variation distance (L_m), and the magnitude of shrinkage or swell is defined by soil heave (Y_m). These parameters are more fully described in Part II.

70. For the center-lift condition, spring definitions included an offset ($D\phi$). This represents the potential soil shrinkage due to moisture changes if no significant loads are applied to the soil, as shown in Figure 8.

71. For the edge-lift condition, the $D\phi$ represents the potential expansion of the soil if no loads are applied. However, the expansive potential

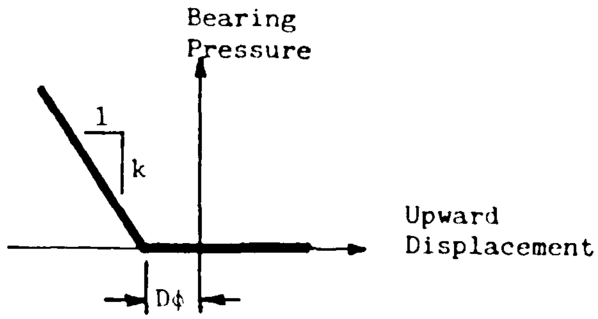


Figure 8. Spring for shrinking soil

was limited to an assumed maximum interface pressure (P_{sw}) between the mat and the soil. This perimeter spring behavior for edge lift is shown in Figure 9.

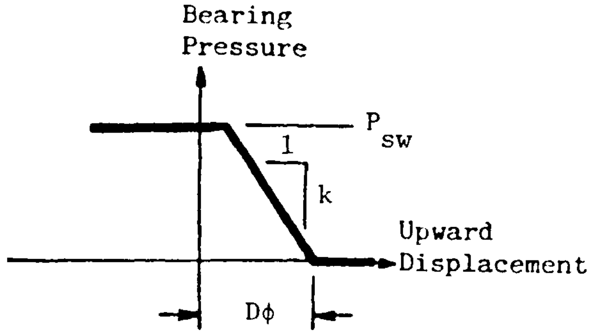


Figure 9. Spring for swelling soil

72. Loading. Loads applied to the beam consisted of a uniform distributed load (p), a concentrated load at the perimeter (P_p), and a concentrated interior load (P_i). The interior load was located at a varying distance (L_i) from the perimeter.

73. Parameter values. A typical range of values was identified for each of the identified parameters, and a baseline (most common) value was selected. The selected parameter values are given in Table 2.

74. Analyses. A computer analysis was performed using the baseline value for each parameter. Additional analyses were then performed by changing the value of a single parameter while retaining all other baseline values. This procedure was followed for both center-lift and edge-lift conditions.

Analysis Results

Numerical results

75. Numerical results of each analysis are presented graphically in Appendix B. Important design results include maximum deflections, moments, and shears. It can be seen that these are affected to differing degrees by variation of each parameter.

Physical analogies

76. A review of the results will indicate that for center lift the end of the beam behaves much as a pure cantilever. For edge lift, the outer portion of the beam behaves similar to a simply supported beam where one support has been raised slightly. Development of design formulas was based on this cantilever and simple support behavior.

Design Formulas

Objective

77. The objective was to develop design formulas which were simple, accurate, rational, and flexible. Flexible indicates that the formulas should be applicable to a wide range of problems. Rational indicates that the formulas should make sense physically to a designer, rather than be a mysterious black box.

Center lift

78. Formulas for center-lift design are included in Part II. The first step is to determine the length of an equivalent cantilever beam. Once this is done the designer uses conventional formulas to determine moments and shears in the cantilever. For deflections, additional adjustments must be made to account for the fact that the support for the cantilever is not truly fixed. The cantilever model makes physical sense to a designer, where determination of the proper length is a black-box formula.

Edge lift

79. Formulas for edge-lift design are included in Part II. The first step is to determine the length of an equivalent simple beam, based on an assumed perimeter deflection. Calculated deflection is used to determine a new equivalent length, and this process continues until assumed deflection converges with calculated deflection. The iterative process increases the complexity of the method, but is unavoidable if accuracy and flexibility of the formulas are to be achieved. Once the equivalent simple beam length is determined, the designer calculates moments and shears by conventional formulas. The simple beam model again makes physical sense to the designer and calculation of edge deflection is based on a rational approach, where determination of the proper length is a black-box formula.

Verification of formulas

80. To demonstrate the accuracy of the formulas, Tables 3 and 4 show comparisons of computer results with formula results for maximum moments and displacement. The comparisons demonstrate sufficient accuracy of the formulas. However, use of parameter values outside the range of those used in the computer analyses or combinations of nonbaseline values for several parameters, will inevitably result in larger differences when comparing formula results to computer solutions. It should be noted that the formulas are

intended only to match the computer results, therefore, adequacy of the formulas is limited by adequacy of the computer model, especially the method used to represent soil behavior. Idealization of soil and structural behavior is fairly crude and should be improved through further, more detailed, investigations.

Table 1

Behavior Checks of Ribbed Mats

Parameter	Units	Minimum	Maximum
k	pci	50	200
Y_m	in.	0.5	3.0
L_m	ft	2	8
I	$in.^4/ft$	750	6,000
P_p	lb/ft	1,000	5,000
P_i	lb/ft	0	5,000
L_i	ft	6	20
w	psf	100	250
P_{sw}	psf	2,000	8,000

Table 2

Parameter Values Used in Computer Analyses

Parameter	Center Lift	Edge Lift
L_m (ft)	2 <u>5</u> 8	2 <u>5</u> 8
Y_m (in.)	0.5 <u>1</u> 2 3	0.5 <u>1</u> 2 3
k (pci)	50 <u>100</u> 200	50 <u>100</u> 200
P_{sw} (psf)	NA	<u>2</u> 4 8
I (1,000 in.^{4})	15 <u>30</u> 60 120	15 <u>30</u> 60 120
s (ft)	12 16 <u>20</u> 24	12 16 <u>20</u> 24
P_p (klf)	1 <u>3</u> 5	0 <u>1</u> 3
P_i (klf)	<u>0</u> 3 5	0 <u>3</u> 5
L_i (ft)	<u>16</u>	6 12 <u>16</u> 20
p (psf)	<u>100</u>	<u>100</u> 250

Note: Baseline values are underlined.

Table 3

Comparison of Center-Lift Results

Parameter	Formulas		Computer		Comparison	
	M (ft-k)	D (in.)	Mc (ft-k)	Dc (in.)	M/Mc	D/Dc
Baseline	13.6	0.324	13.2	0.32	1.03	1.01
k = 50	13.6	0.413	13.2	0.41	1.03	1.01
k = 200	13.6	0.261	13.2	0.26	1.03	1.00
Y_m = 0.5	12.5	0.284	12.5	0.27	1.00	1.05
Y_m = 2.0	14.9	0.374	15.6	0.36	0.96	1.04
Y_m = 3.0	15.7	0.408	16.0	0.39	0.98	1.05
L_m = 2	9.6	0.205	9.2	0.19	1.04	1.08
L_m = 8	17.7	0.507	17.1	0.54	1.04	0.94
I/s = 0.75	12.1	0.435	12.5	0.43	0.97	1.01
I/s = 3	15.3	0.251	15.9	0.23	0.96	1.09
I/s = 6	17.3	0.203	17.4	0.20	0.99	1.02
P_p = 1	6.0	0.188	6.2	0.15	0.97	1.25
P_p = 5	20.7	0.473	20.8	0.47	1.00	1.01

Table 4

Comparison of Edge-Lift Results

Parameter	Formulas		Computer		Comparison	
	M (ft-k)	D (in.)	Mc (ft-k)	Dc (in.)	M/Mc	D/Dc
Baseline	12.8	0.51	11.8	0.55	1.08	0.93
$Y_m = 0.5$	9.4	0.27	7.3	0.26	1.29	1.04
$Y_m = 2.0$	16.9	0.94	18.2	1.00	0.93	0.94
$Y_m = 3.0$	19.3	1.35	22.5	1.38	0.86	0.98
$L_m = 2$	6.6	0.17	5.7	0.17	1.16	1.00
$L_m = 8$	14.7	0.66	13.7	0.66	1.07	1.00
I/s = 0.75	7.8	0.57	7.1	0.60	1.10	0.95
I/s = 3	18.6	0.45	17.5	0.46	1.06	0.98
I/s = 6	23.9	0.41	24.5	0.39	0.98	1.05
$P_p = 0$	14.7	0.66	13.7	0.66	1.07	1.00
$P_p = 3$	9.1	0.27	8.2	0.26	1.11	1.04
$P_i = 0$	7.6	0.57	7.2	0.57	1.06	1.00
$P_i = 5$	14.6	0.49	13.7	0.53	1.07	0.92
$L_i = 6$	12.3	0.40	12.2	0.34	1.01	1.18
$L_i = 12$	15.4	0.47	14.7	0.48	1.05	0.98
$L_i = 20$	9.4	0.54	8.3	0.54	1.13	1.00
p = 250	13.6	0.36	10.4	0.42	1.31	0.86
$P_{sw} = 4$	15.2	0.72	13.3	0.65	1.14	1.11
$P_{sw} = 8$	16.4	0.85	13.5	0.68	1.21	1.25

APPENDIX A: DESIGN EXAMPLE

DESIGN EXAMPLE

(RIBBED MAT DESIGN IN EXPANSIVE SOIL)

1. SOIL DATA (Part I, paragraph 10)

q_a = 2,000 psf

P_{sw} = (see Appendix A, paragraph 14)

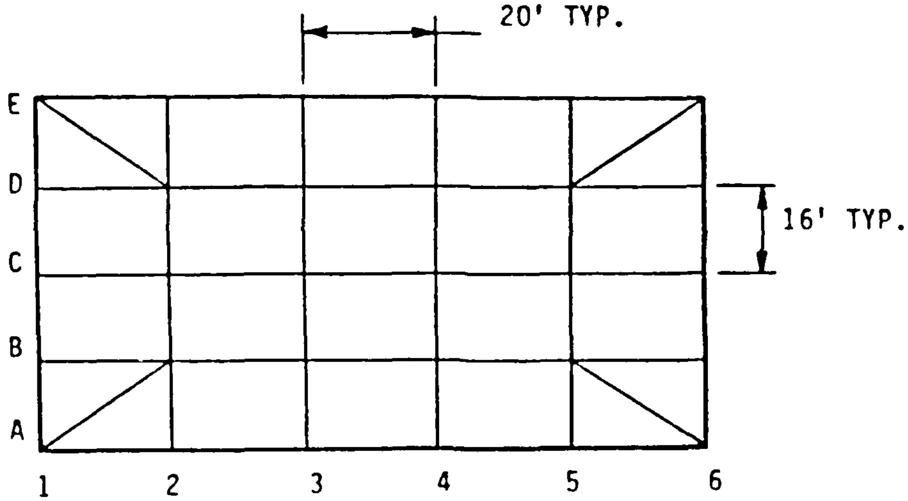
k = 100 pci

L_m = 6 ft

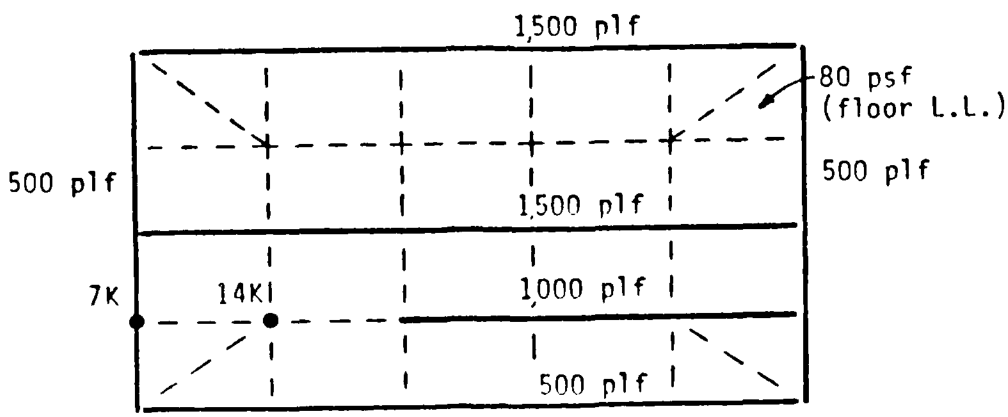
Y_m = 1.5 in. for center lift

Y_m = 1.0 in. for edge lift

2. FOUNDATION PLAN (Part I, paragraph 13)



3. LOADS



4. BEARING DESIGN FOR RIBS (Part I, paragraph 4)

Maximum wall load (P) = 1,500 plf

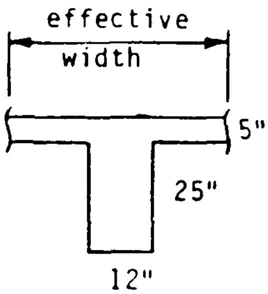
Width $\geq P/q_a$ = 1,500/2,000 = 0.75 ft

Use 12-inch wide ribs (minimum)

5. INTERIOR RIB PROPERTIES (Part III, paragraphs 41 through 47)

E_c = 3,320,000 psi

(effective flange width per ACI 318, section 8.10.2 For "span length" use $4L_c$ for center lift or L_e for edge lift)



Let I_r = 36,000 in.^{4} for center lift

I_r = 24,000 in.^{4} for edge lift

(ref. ACI 318, section 9.5.2.3, verify I_r after calculating M)

$I = I_r/S$ (in.^{4}/ft):

Rib spacing	16 ft	20 ft
Center lift	2,250	1,800
Edge lift	1,500	1,200

6. CENTER-LIFT DESIGN - RIB E3/C3

6.1 Loads (Part III, paragraphs 41 through 47)

slab weight = 150 pcf × 5/12 ft = 62 psf

w = DL + LL = 62 + 80 = 142 psf

rib weight = 150 pcf × 2.5 ft × 1.0 ft = 375 plf

Pp = rib + wall = 375 + 1,500 = 1,875 plf

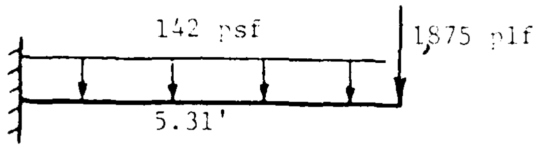
6.2 Equivalent cantilever (Part II, paragraphs 26 through 30)

$L_o = 2.3 + 0.4\ L_m = 2.3 + (0.4 \times 6) = 4.7$ ft

$C = 0.8\ Y_m^{0.12}\ I^{0.16}/P_p^{0.12}$

$$C = 0.8 \times 1.5^{0.12} \times 1,800^{0.16}/1,875^{0.12} = 1.13$$

$$L_c = L_o\ C = 4.7 \times 1.13 = 5.31\ \text{ft}$$



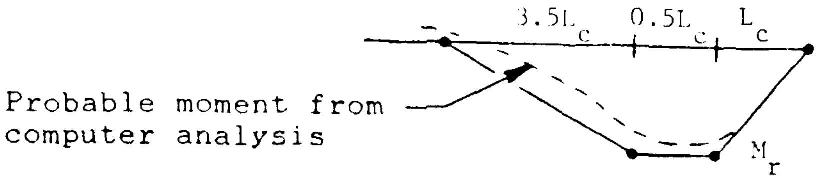
6.3 Moment (Part II, paragraph 27)

$$M = P_p\ L_c + 1/2\ w\ L_c^2$$

$$M = 1,875 \times 5.31 + 1/2 \times 142 \times 5.31^2 = 12,000\ \text{ft-lb/ft}$$

$$M_r = M \times S = 12,000 \times 20 = 240,000\ \text{ft-lb/rib}$$

Design moments:

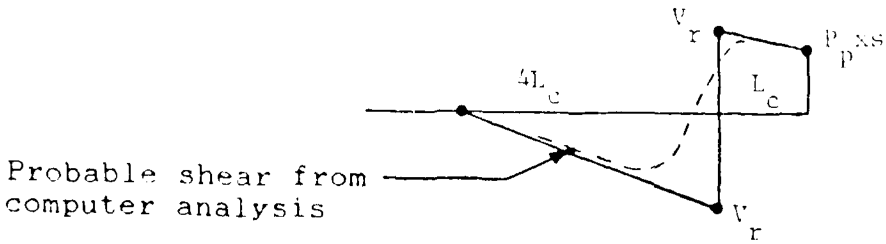


6.4 Shear (Part II, paragraph 28)

$$V = P_p + w\ L_c = 1,875 + 142 \times 5.31 = 2,630\ \text{lb/ft}$$

$$V_r = V \times S = 2,630 \times 20 = 52,600\ \text{lb/rib}$$

Design shears:



6.5 Reinforcing in rib (Part I, paragraphs 8 and 15)

$$A_s = (M_r/ad)/1.33$$

$A_s = 240/(1.76 \times 28 \times 1.33) = 3.66\ \text{in.}^2$ (top) use 3 #10 bars

$v = V_r/bd = 52{,}600/(12 \times 28) = 157$ psi

$vc = (1.1\sqrt{f'_c})1.33 = 80$ psi

$A_v = (v - vc)b\ s/(fs\ 1.33)$

$A_v = (157 - 80)12 \times 12/(24{,}000 \times 1.33) = 0.35$ in.^{2}/ft

use #4 stirrups @ 12 in.

6.6 Deflection (Part II, paragraph 29)

$\theta = M^{1.4}/9{,}800\ I\ k^{0.5}$

$\theta = 12{,}000^{1.4}/(9{,}800 \times 1{,}800 \times 100^{0.5}) = 0.0029$ radians

$D = 0.11 + 12\ L_c\ \theta = 0.11 + 12 \times 5.31 \times 0.0029 = 0.29$ in.

$D/4L_c = 0.29/(4 \times 5.31 \times 12) = 1/879$ O.K.

7. EDGE-LIFT DESIGN - RIB A2/C2

7.1 Loads

$w = 142$ psf (same as above)

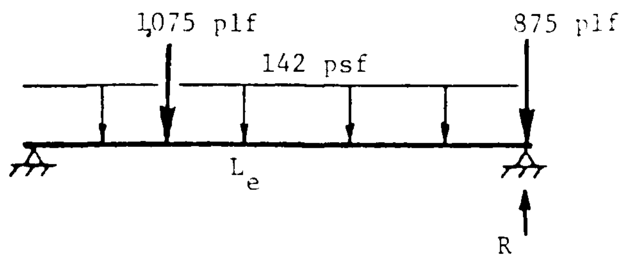
P_p = rib + wall = 375 + 500 = 875 plf

P_i = rib + wall* = 375 + 700 = 1,075 plf

* equivalent wall load = column load/rib spacing
14,000/20 = 700 plf (Part III, paragraph 50)

$L_i = 16$ ft

7.2 Equivalent simple beam (Part III, paragraph 60)



7.3 Deflection (Part II, paragraph 32)

$L_e = 7.5\ I^{0.17}\ Li^{0.37}\ D^{0.12}/w^{0.07}\ Pi^{0.11}$

$L_e = 7.5 \times 1{,}200^{0.17} \times 16^{0.37} \times D^{0.12}/142^{0.07} \times 1{,}075^{0.11}$

$L_e = 22.9\ D^{0.12}$

assume D = 0.50 in. (somewhat less than $Y_m = 1.0$ in.)

$L_e = 22.9 \times 0.50^{0.12} = 21.1$ ft

$R = P_p + 1/2\ w\ L_e + P_i(L_e - L_i)/L_e$

$R = 875 + (142 \times 21.1)/2 + 1{,}075(21.1 - 16.0)/21.1 = 2{,}633$ plf

from heave/pressure curve (paragraph 14), for D = 0.50 find $P_{sw} = 2{,}000$ psf

$L_b = 1.1(R/P_{sw}) = 1.1(2{,}633/2{,}000) = 1.45$ ft

$D = Y_m(L_m - L_b)^2/L_m^2$

$D = 1.0(6.0 - 1.45)^2/6.0^2 = 0.575$ in. $\neq$ 0.50 in. assumed!

assume D = 0.54 in.

$L_e = 22.9 \times 0.54^{0.12} = 21.3$ ft

$R = P_p + 1/2\ w\ L_e + P_i(L_e - L_i)/L_e$

$R = 875 + (142 \times 21.3)/2 + 1{,}075(21.3 - 16.0)/21.3 = 2{,}655$ plf

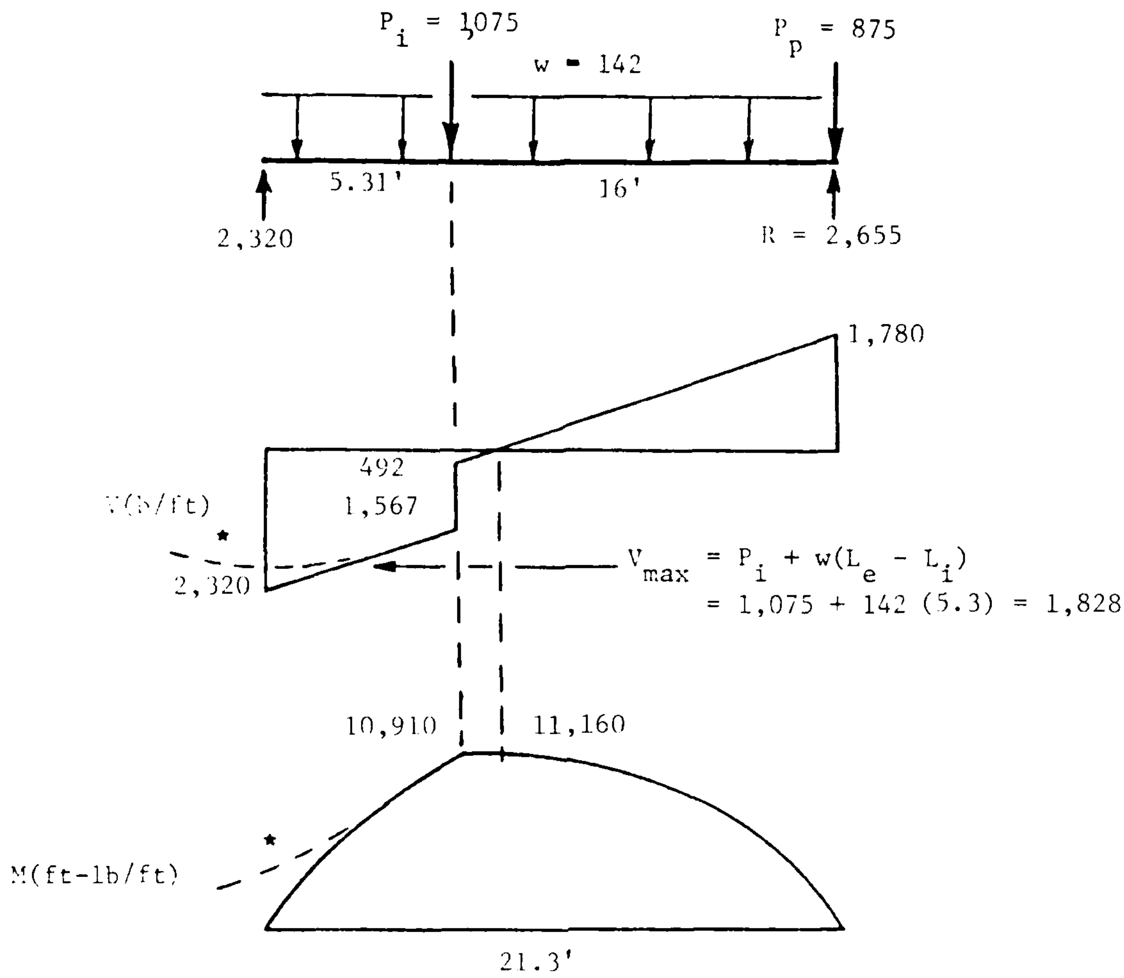
from heave/pressure curve, for D = 0.54 find $P_{sw} = 1{,}800$ psf

$L_b = 1.1(R/P_{sw}) = 1.1(2{,}655/1{,}800) = 1.62$ ft

$D = 1.0(6.0 - 1.62)^2/6.0^2 = 0.533$ in. CONVERGED!

$D/L_e = 0.54/(21.3 \times 12) = 1/473$ O.K. for nonbrittle walls

7.4 Moment and shear (Part II, paragraphs 33 and 34)



* probable shear and moment from computer analysis, note that calculated V = 2,320 lb will not occur, due to the effects of distributed support from the soil

8. EDGE-LIFT DESIGN - RIB E4/C4

8.1 Loads

w = 142 psf (same as above)

P_p = 1,875 plf (same as rib E3/C3)

L_i = 32 ft (wall along rib C1/C6)

8.2 Deflection

since $L_i > L_e$ use:

$L_e = 10.5\ I^{0.17}\ D^{0.12}/w^{0.07}$ (Part II, paragraph 35)

$L_e = 10.5 \times 1{,}200^{0.17} \times D^{0.12}/142^{0.07} = 24.77\ D^{0.12}$

assume D = 0.48 in.

$L_e = 24.77 \times 0.48^{0.12} = 22.7$ ft

$R = P_p + 1/2\ w\ L_e = 1{,}875 + (142 \times 22.7)/2 = 3{,}485$ plf

from heave/pressure curve, for D = 0.48 find $P_{sw} = 2{,}100$ psf

$L_b = 1.1(R/P_{sw}) = 1.1(3{,}485/2{,}100) = 1.825$ ft

$D = Y_m(L_m - L_b)^2/L_m^2$

$D = 1.0(6.0 - 1.825)^2/6.0^2 = 0.484$ in. CONVERGED!

8.3 Find shears and moments by statics, similar to rib A2/C2.

9. CENTER-LIFT DESIGN - RIB C1/C3

9.1 Loads

w = slab + LL + wall* = 62 + 80 + 94 = 236 psf

* wall = wall load/rib spacing = 1,500/16 = 94 psf (Part III, paragraph 50)

P_p = rib + wall = 375 + 500 = 875 plf

9.2 Equivalent cantilever

$L_o = 2.3 + 0.4\ L_m = 2.3 + (0.4 \times 6) = 4.7$ ft

$C = 0.8\ Y_m^{0.12}\ I^{0.16}/P_p^{0.12}$

$C = 0.8 \times 1.5^{0.12} \times 2{,}250^{0.16}/875^{0.12} = 1.28$

$L_c = L_o\ C = 4.7 \times 1.28 = 6.02$ ft

9.3 Moment

$M = P_p\ L_c + 1/2\ w\ L_c^2$

$M = 875 \times 6.02 + (236 \times 6.02^2)/2 = 9{,}544$ ft-lb/ft

$M_r = M \times S = 9{,}544 \times 16 = 153{,}000$ ft-lb/rib

9.4 Shear

$$V = P_p + w\,L_c = 875 + (236 \times 6.02) = 2{,}296 \text{ plf}$$

$$V_r = V \times S = 2{,}296 \times 16 = 36{,}700 \text{ lb/rib}$$

9.5 Deflection

$$\theta = M^{1.4}/9{,}800\ I\ k^{0.5}$$

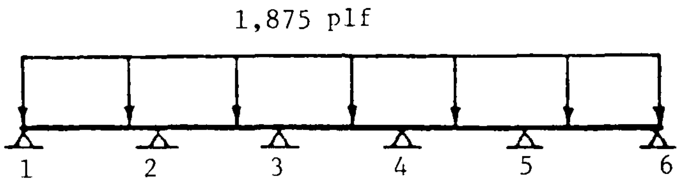
$$\theta = 9{,}544^{1.4}/9{,}800 \times 2{,}250 \times 100^{0.5} = 0.0017 \text{ radian}$$

$$D = 0.11 + 12\ L_c\ \theta = 0.11 + (12 \times 6.02 \times 0.0017) = 0.23 \text{ in.}$$

10. CENTER-LIFT DESIGN - PERIMETER RIB E1/E6 (Part II, paragraph 36)

10.1 Span between transverse ribs

P_p = 1,875 plf (from calculations for rib E3/C3)

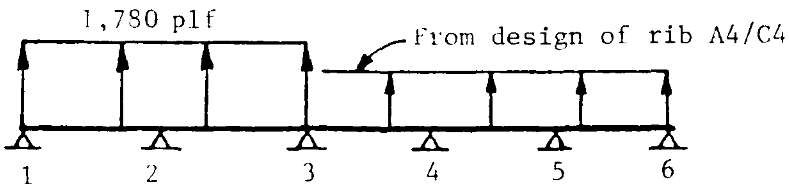


10.2 Analyze by conventional methods

11. EDGE-LIFT DESIGN - PERIMETER RIB A1/A3 (Part II, paragraph 37)

11.1 Span between transverse ribs for net upward force (from calculations on rib A2/C2)

$R - P_p$ = 2,655 - 875 = 1,780 plf (upward)



11.2 Analyze by conventional methods

12. CENTER-LIFT DESIGN - DIAGONAL RIB A1/B2 (Part II, paragraph 38)

12.1 Provide the larger shear and moment capacity of rib B1/B2 or rib A2/B2.

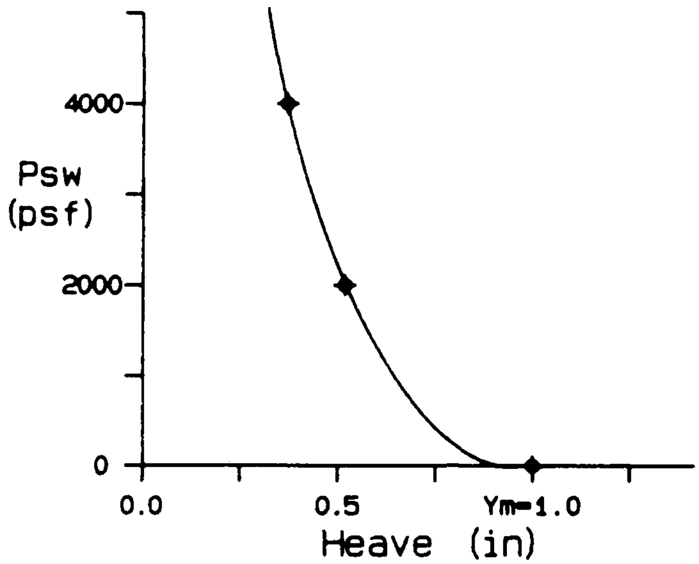
13. RIB D3/D4 (Part I, paragraph 15)

13.1 Interior rib with no wall or column loads

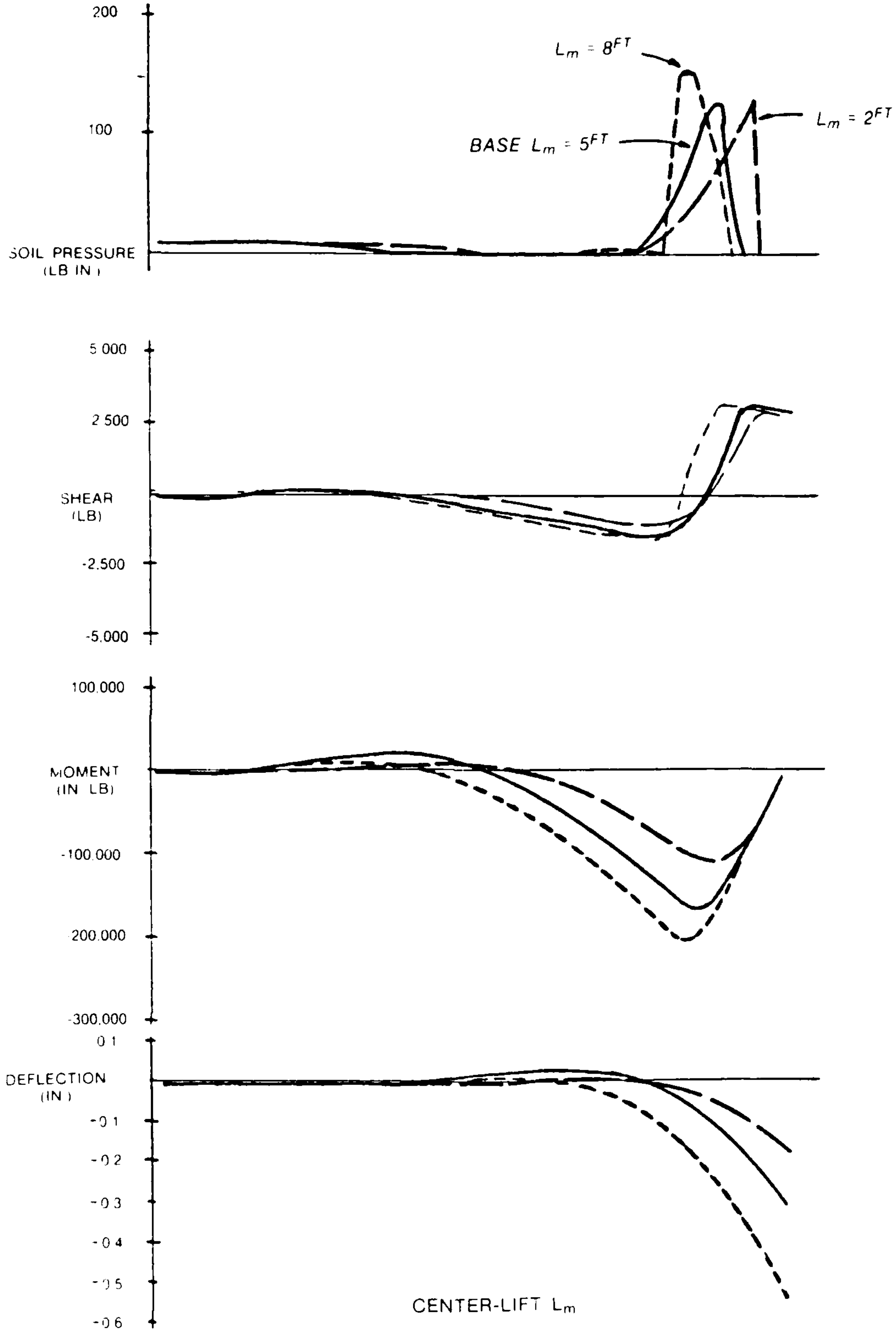
$A_s \geq 0.005\ A_g = 0.005 \times 12 \times 30 = 1.80\ \text{in.}^2$ (top and bottom)

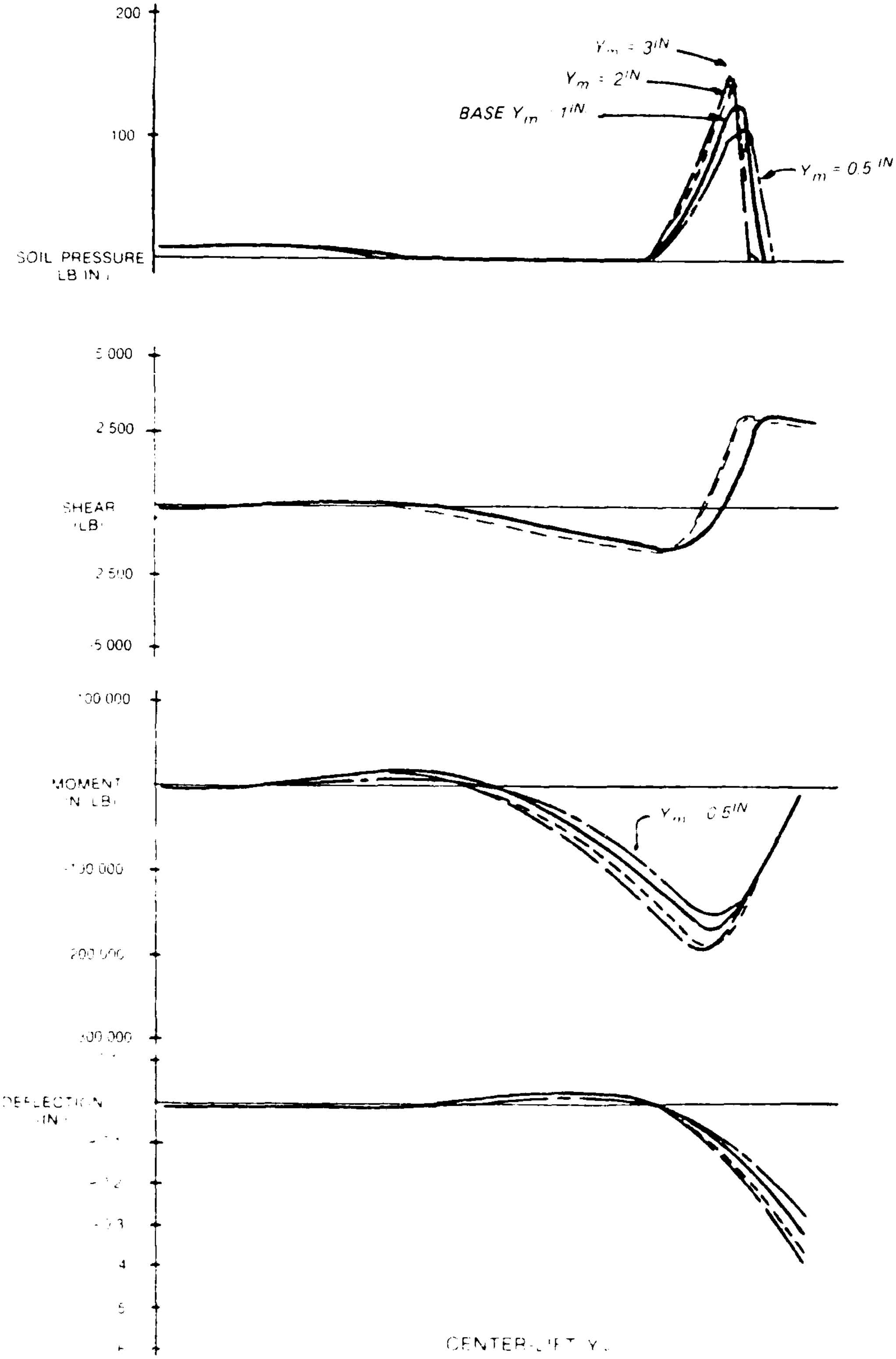
This is the typical minimum reinforcement for the full length of all ribs.

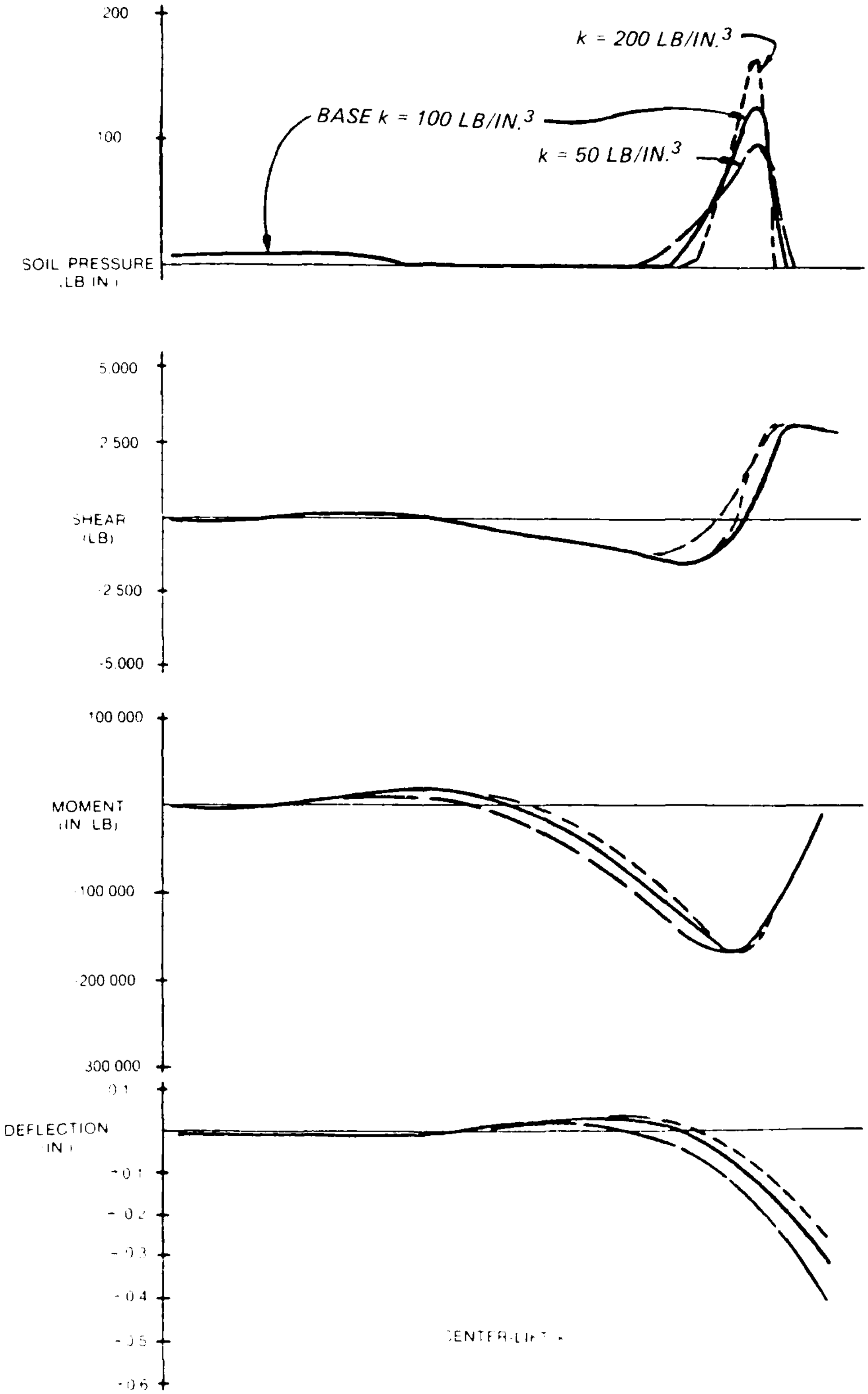
14. HEAVE VERSUS SWELL PRESSURE CURVE (Part III, paragraph 45)



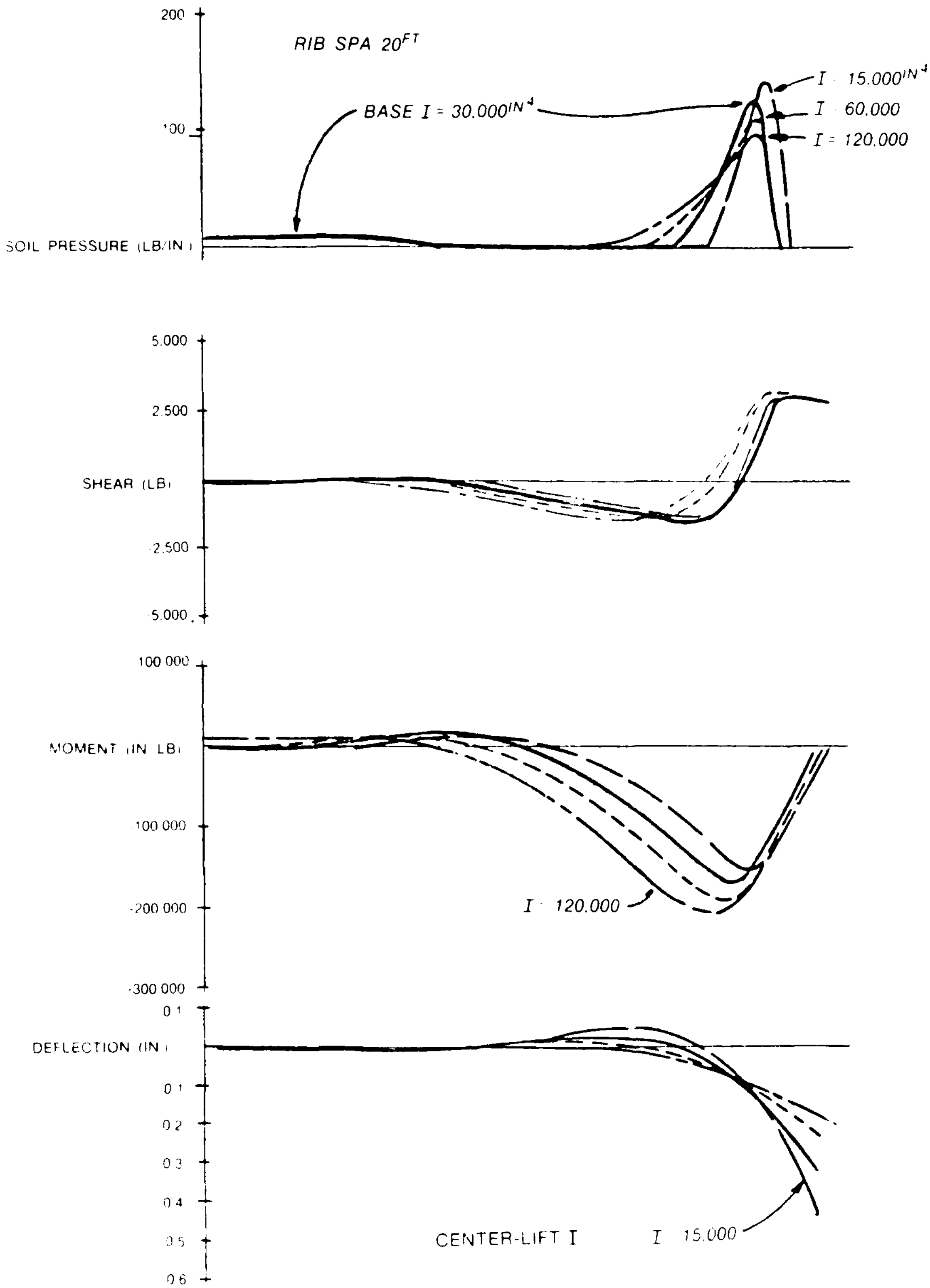
APPENDIX B: NUMERICAL RESULTS OF ANALYSIS

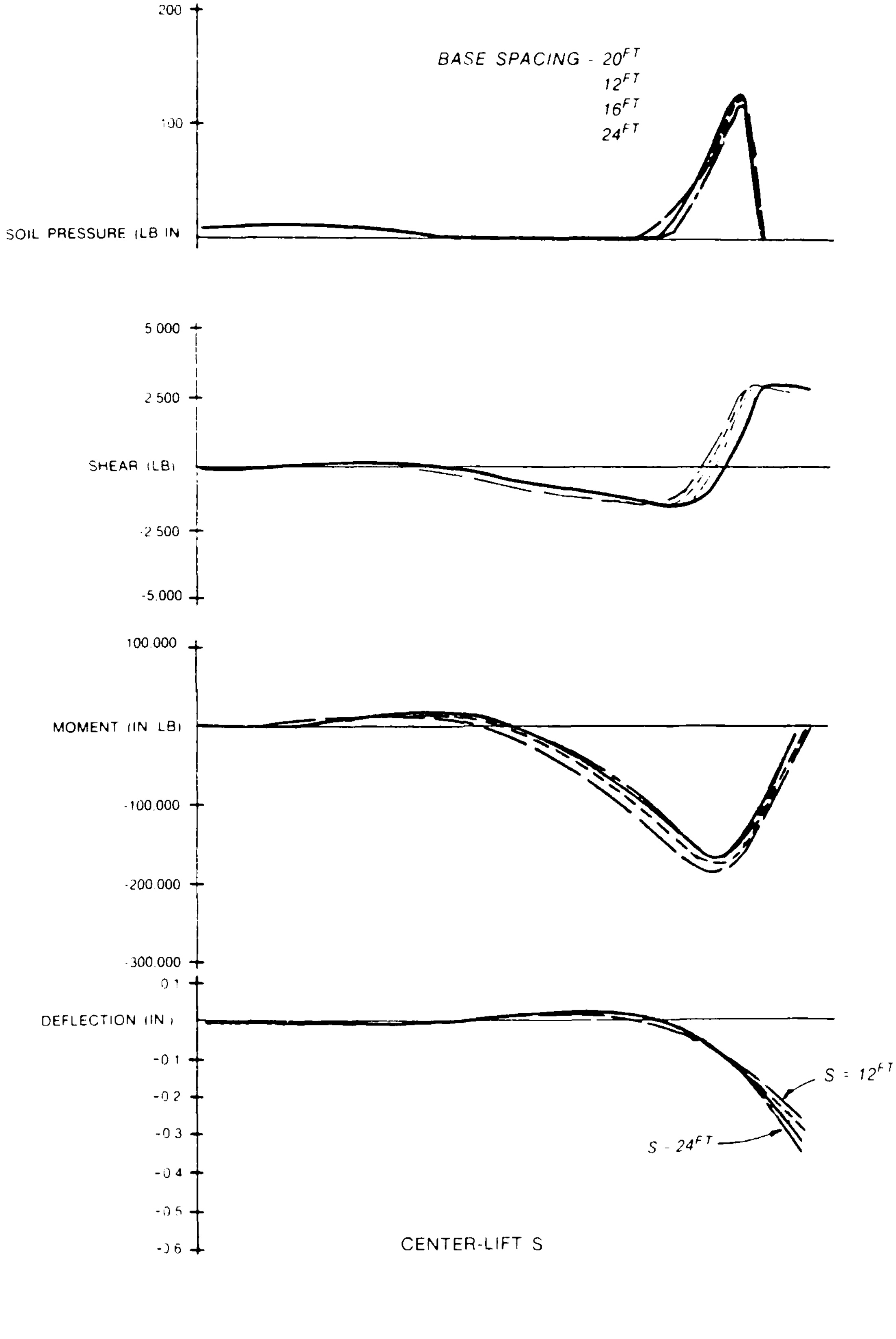

200
100
SOIL PRESSURE
(LB IN)
L_m = 8 FT
BASE L_m = 5 FT
L_m = 2 FT
5,000
2,500
SHEAR
(LB)
-2,500
-5,000
100,000
MOMENT
(IN LB)
-100,000
200,000
-300,000
0.1
DEFLECTION
(IN)
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
CENTER-LIFT L_m

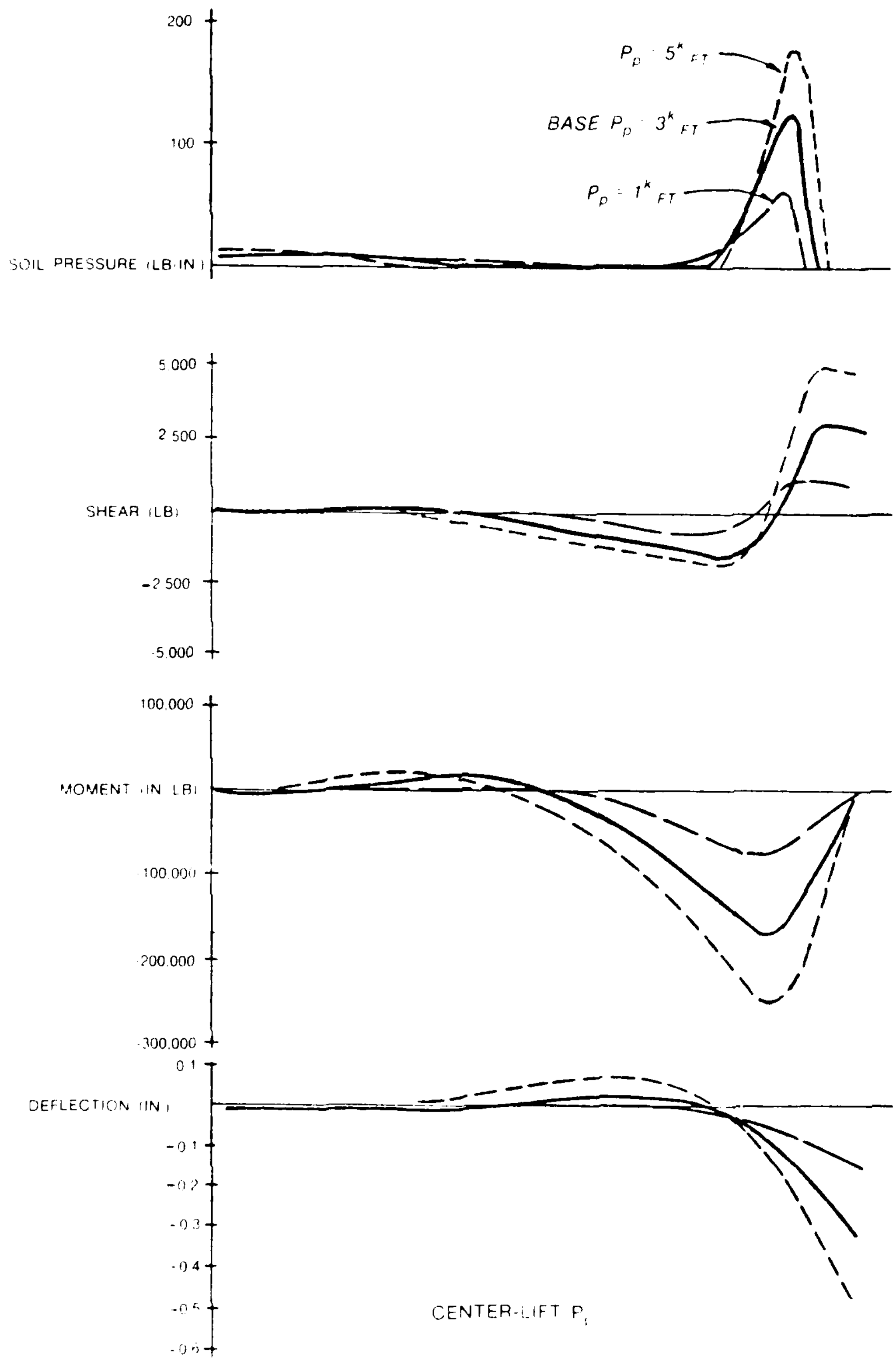

200
100
SOIL PRESSURE
LB IN
$Y_m = 3^{IN}$
$Y_m = 2^{IN}$
BASE Y_m 1^{IN}
$Y_m = 0.5^{IN}$
2 500
SHEAR
LB
2 500
5 000
100 000
MOMENT
IN LB
Y_m 0.5^{IN}
4
5



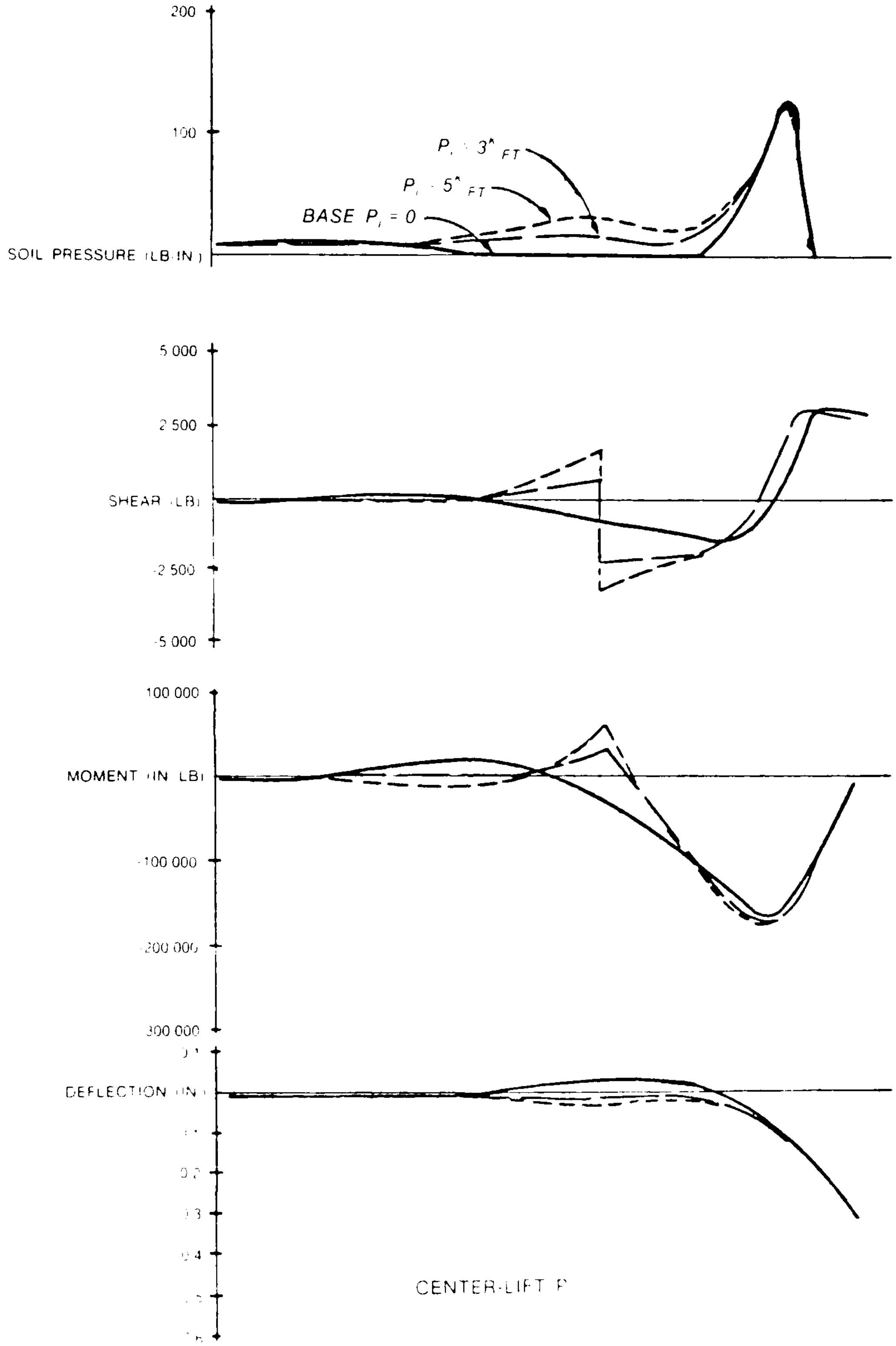
200
100
SOIL PRESSURE (LB/IN.)
k = 200 LB/IN.^{3}
BASE k = 100 LB/IN.^{3}
k = 50 LB/IN.^{3}
5,000
2,500
SHEAR (LB)
-2,500
-5,000
100,000
MOMENT (IN.-LB)
-100,000
-200,000
-300,000
0.1
DEFLECTION (IN.)
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6


200
RIB SPA 20FT
I 15.000IN4
BASE I = 30.000IN4
I 60.000
100
I = 120.000
SOIL PRESSURE (LB/IN)
5.000
2.500
SHEAR (LB)
2.500
5.000
100 000
MOMENT (IN LB)
100 000
I 120.000
200 000
300 000
0 1
DEFLECTION (IN)
0 1
0 2
0 3
0 4
0 5
0 6
CENTER-LIFT I
I 15.000

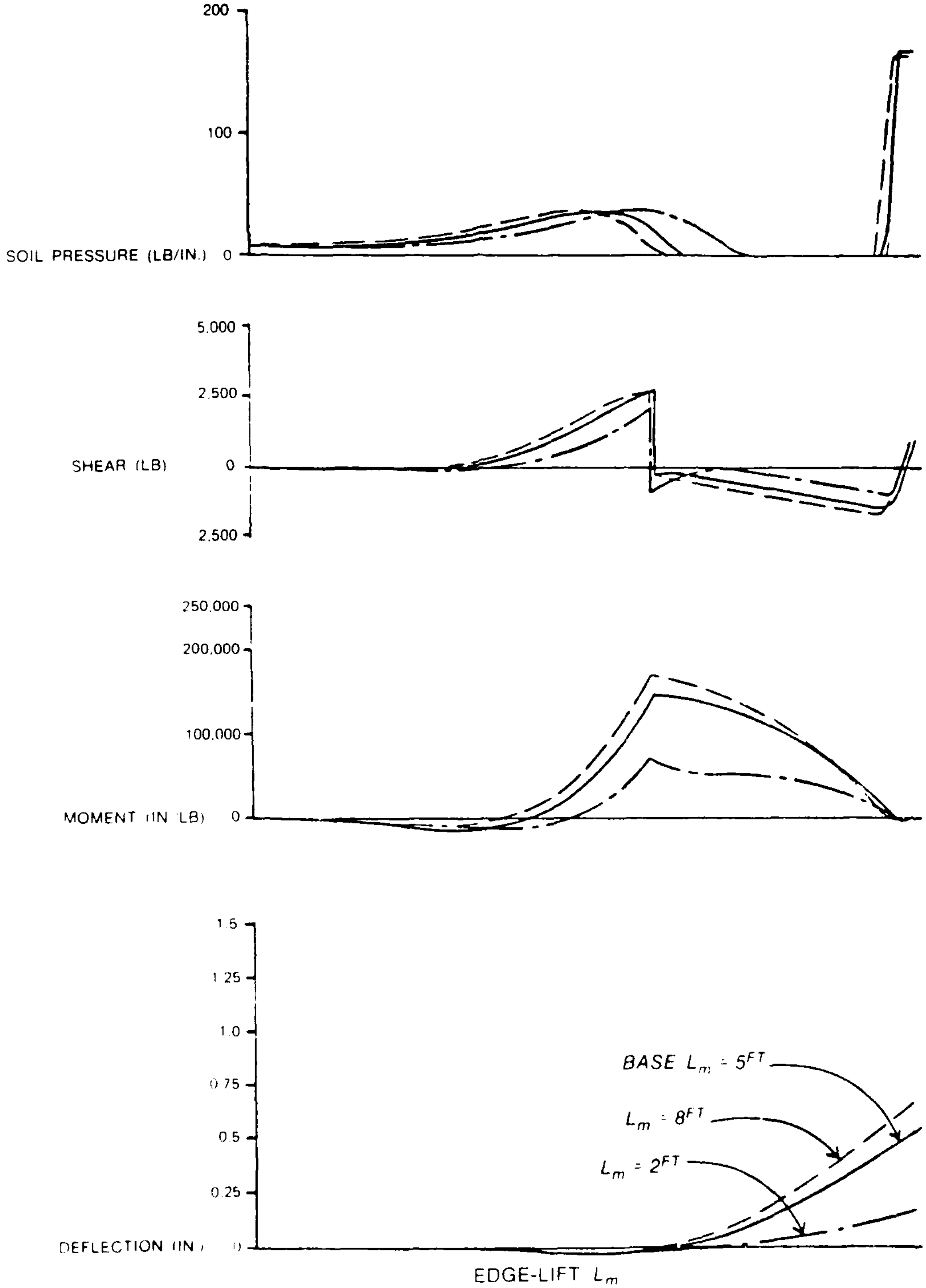

200
BASE SPACING - 20FT
12FT
16FT
24FT
100
SOIL PRESSURE (LB IN
5 000
2 500
SHEAR (LB)
-2 500
-5,000
100,000
MOMENT (IN LB)
-100,000
-200,000
-300,000
0.1
DEFLECTION (IN.)
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
S = 12FT
S - 24FT
CENTER-LIFT S

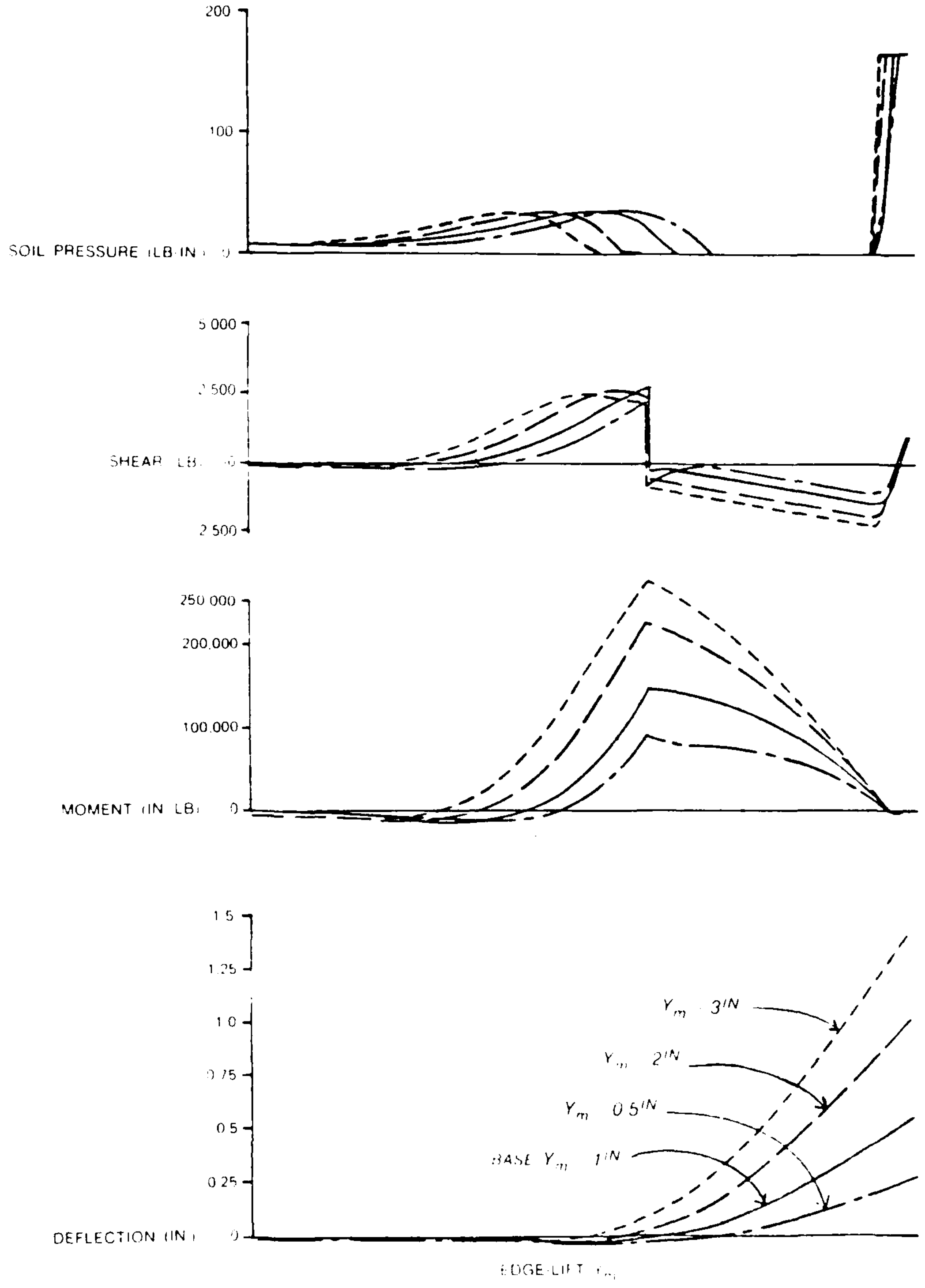


CENTER-LIFT P_p

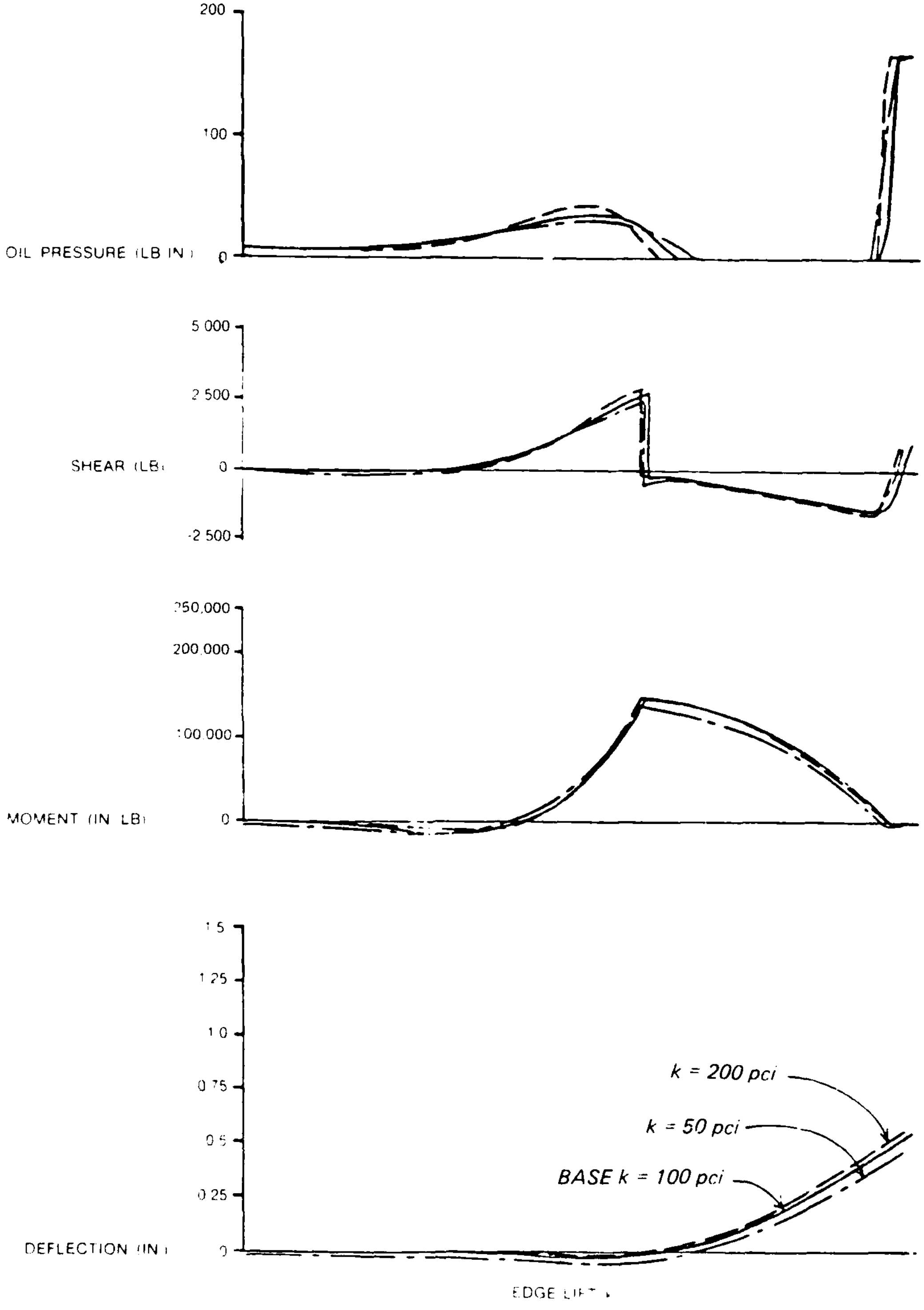


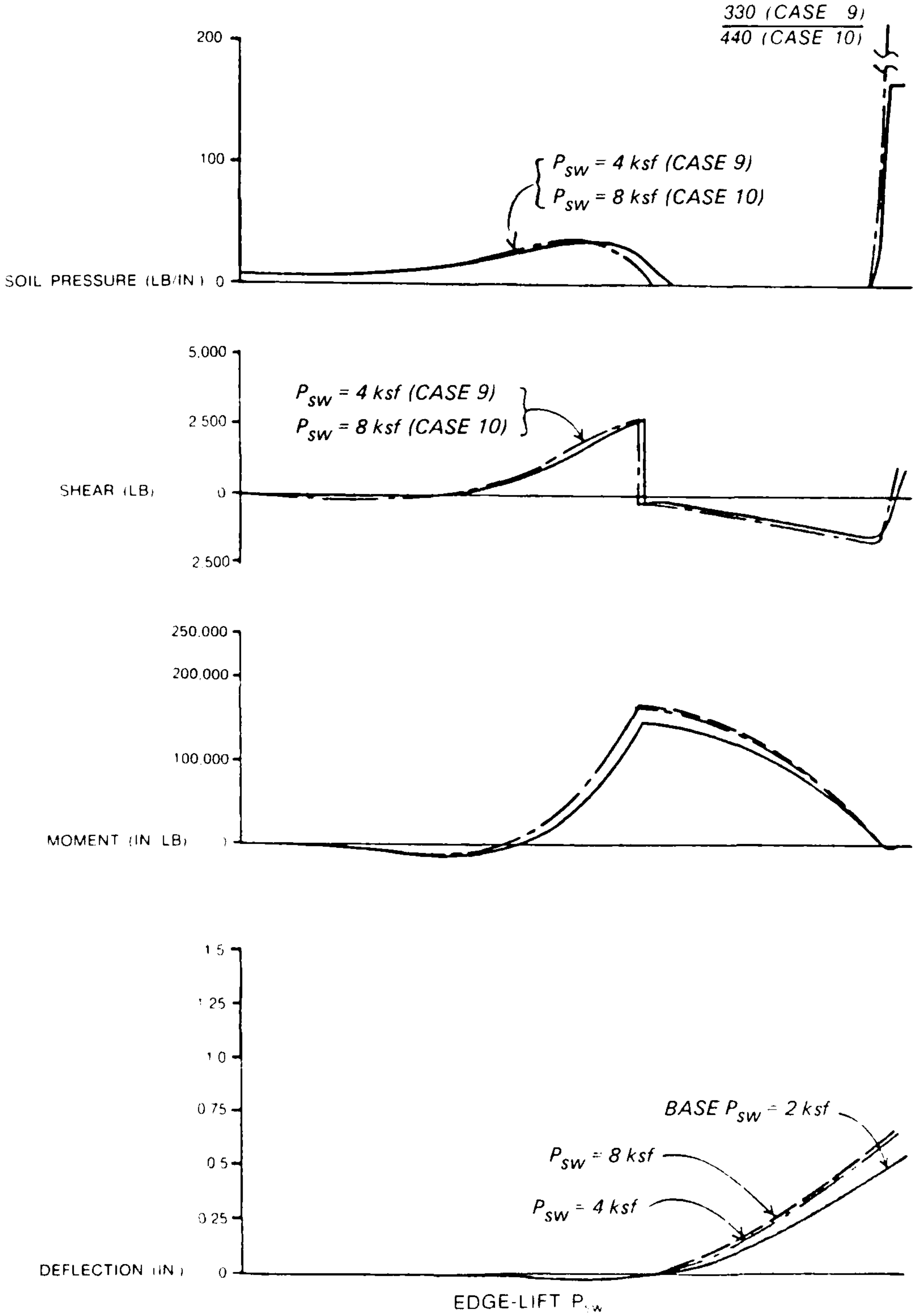
CENTER-LIFT P

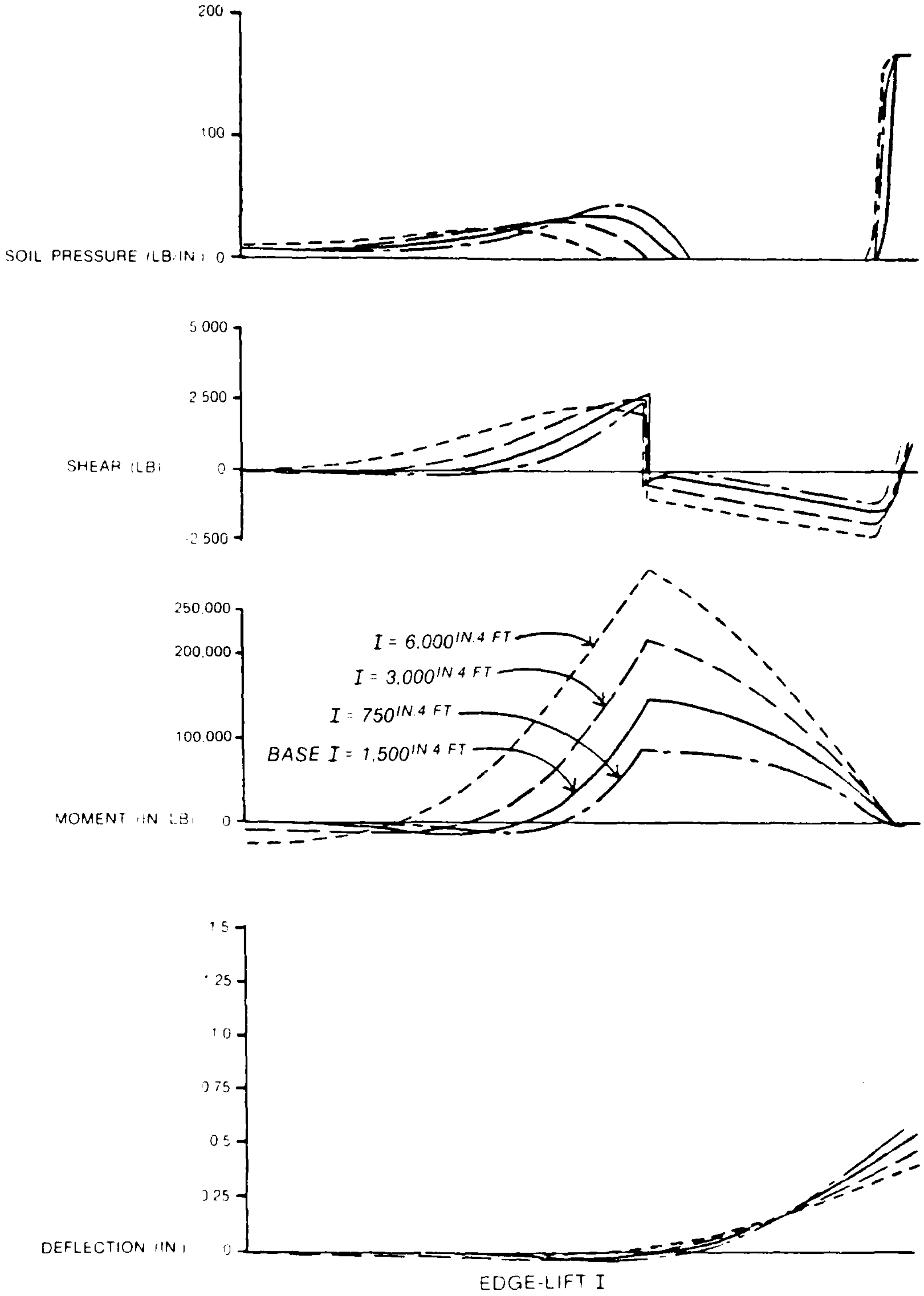

200
100
SOIL PRESSURE (LB/IN.) 0
5,000
2,500
SHEAR (LB) 0
2,500
250,000
200,000
100,000
MOMENT (IN LB) 0
1.5
1.25
1.0
0.75
0.5
0.25
DEFLECTION (IN.) 0
BASE $L_m = 5^{FT}$
$L_m = 8^{FT}$
$L_m = 2^{FT}$
EDGE-LIFT L_m

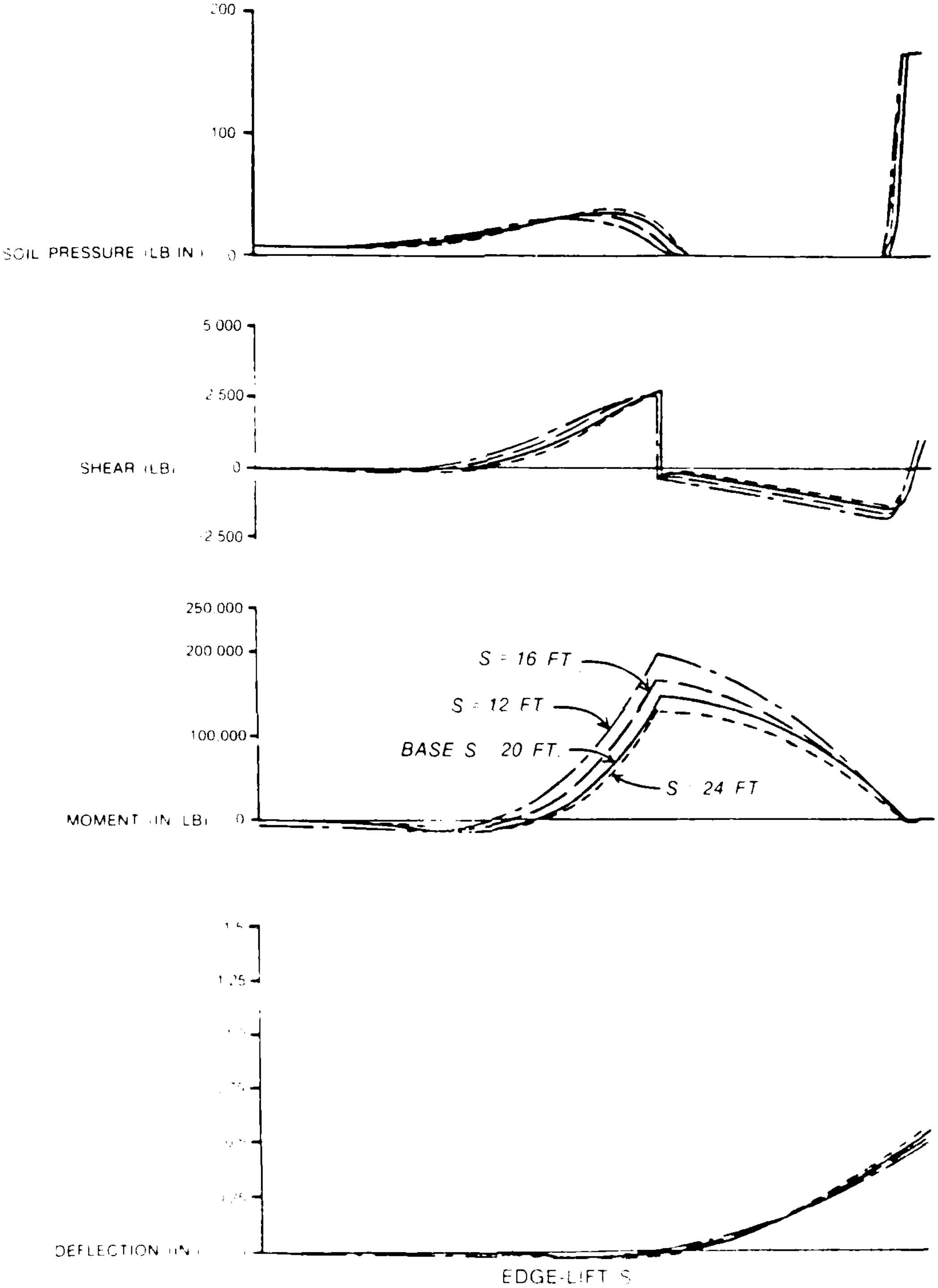


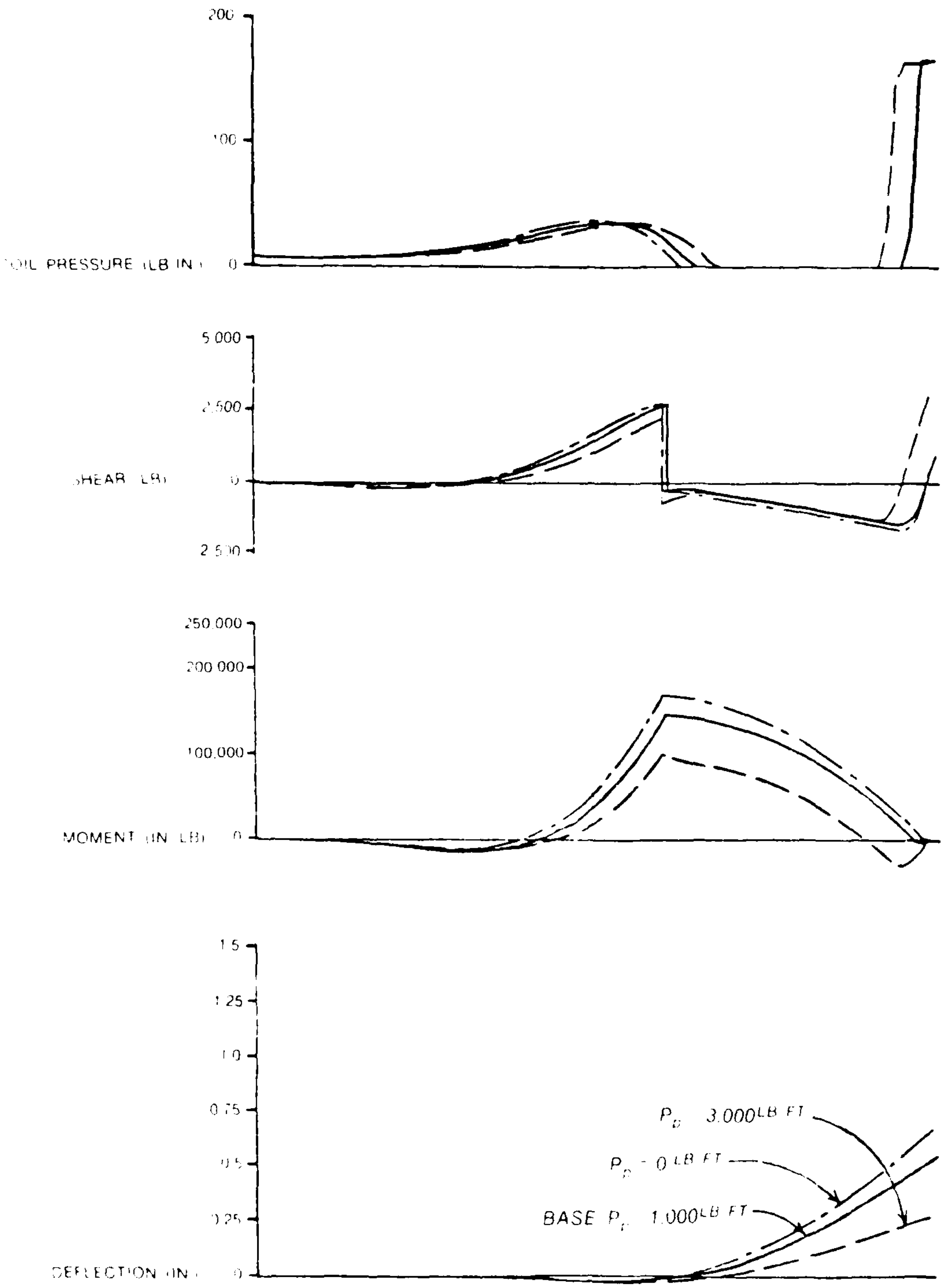
200
100
SOIL PRESSURE
5 000
SHEAR
2 500
250 000
200,000
100,000
MOMENT
0
1.5
1.25
1.0
0.75
0.5
0.25
DEFLECTION (IN)
BASE
EDGE-LIFT


200
100
OIL PRESSURE (LB IN)
0
5 000
2 500
SHEAR (LB)
0
-2 500
250,000
200,000
100,000
MOMENT (IN LB)
0
1 5
1 25
1 0
0 75
0 5
0 25
DEFLECTION (IN)
0
k = 200 pci
k = 50 pci
BASE k = 100 pci
EDGE LIFT

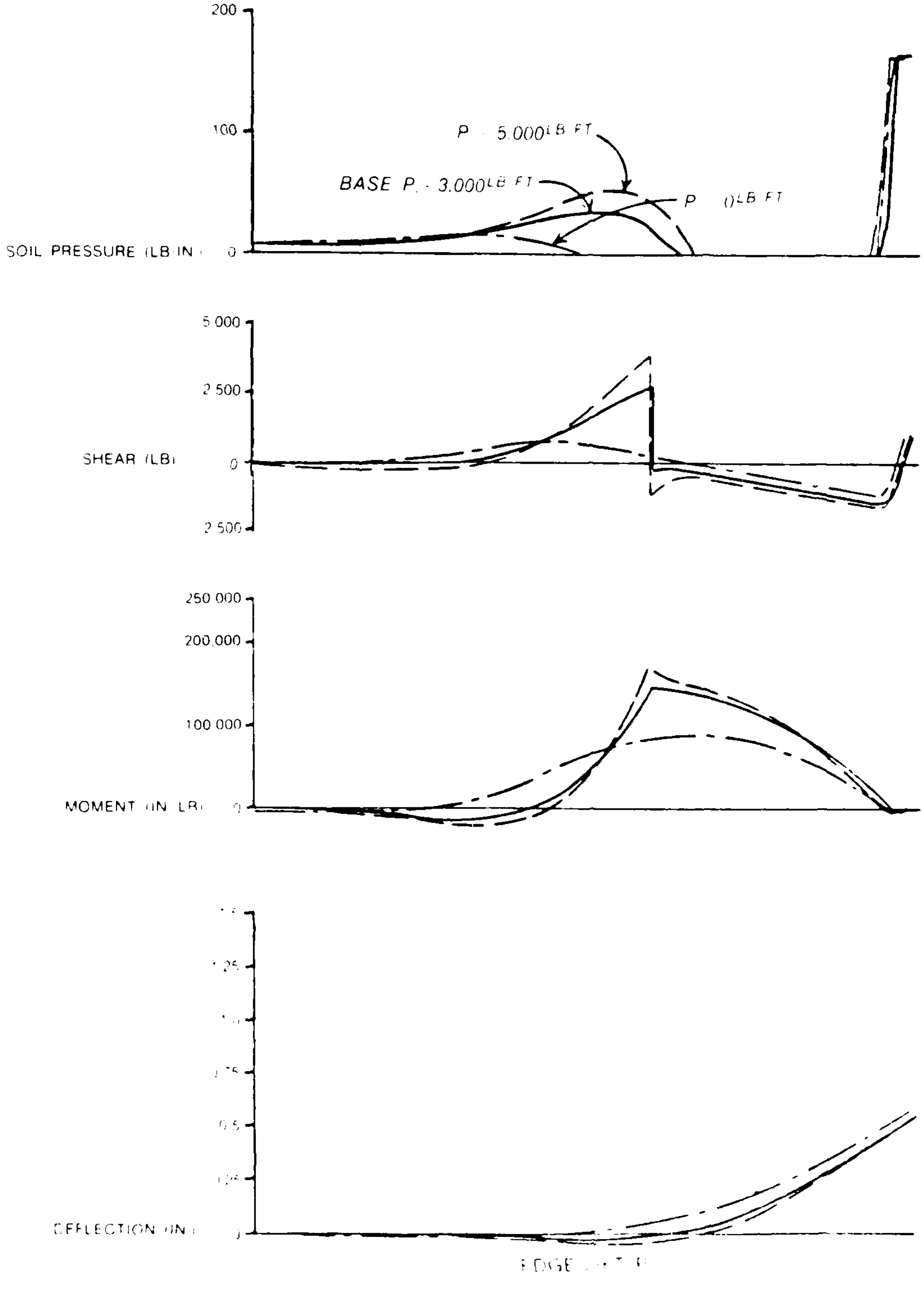

330 (CASE 9)
440 (CASE 10)
200
100
P_{sw} = 4 ksf (CASE 9)
P_{sw} = 8 ksf (CASE 10)
SOIL PRESSURE (LB/IN) 0
5,000
2 500
0
2 500
SHEAR (LB)
P_{sw} = 4 ksf (CASE 9)
P_{sw} = 8 ksf (CASE 10)
250,000
200,000
100,000
MOMENT (IN LB)
1.5
1.25
1.0
0.75
0.5
0.25
0
DEFLECTION (IN)
BASE P_{sw} = 2 ksf
P_{sw} = 8 ksf
P_{sw} = 4 ksf
EDGE-LIFT P_{sw}


200
100
SOIL PRESSURE (LB/IN) 0
5 000
2 500
SHEAR (LB) 0
-2 500
250,000
200,000
100,000
MOMENT (IN-LB) 0
I = 6,000 IN.4 FT
I = 3,000 IN.4 FT
I = 750 IN.4 FT
BASE I = 1,500 IN.4 FT
1.5
1.25
1.0
0.75
0.5
0.25
DEFLECTION (IN) 0
EDGE-LIFT I

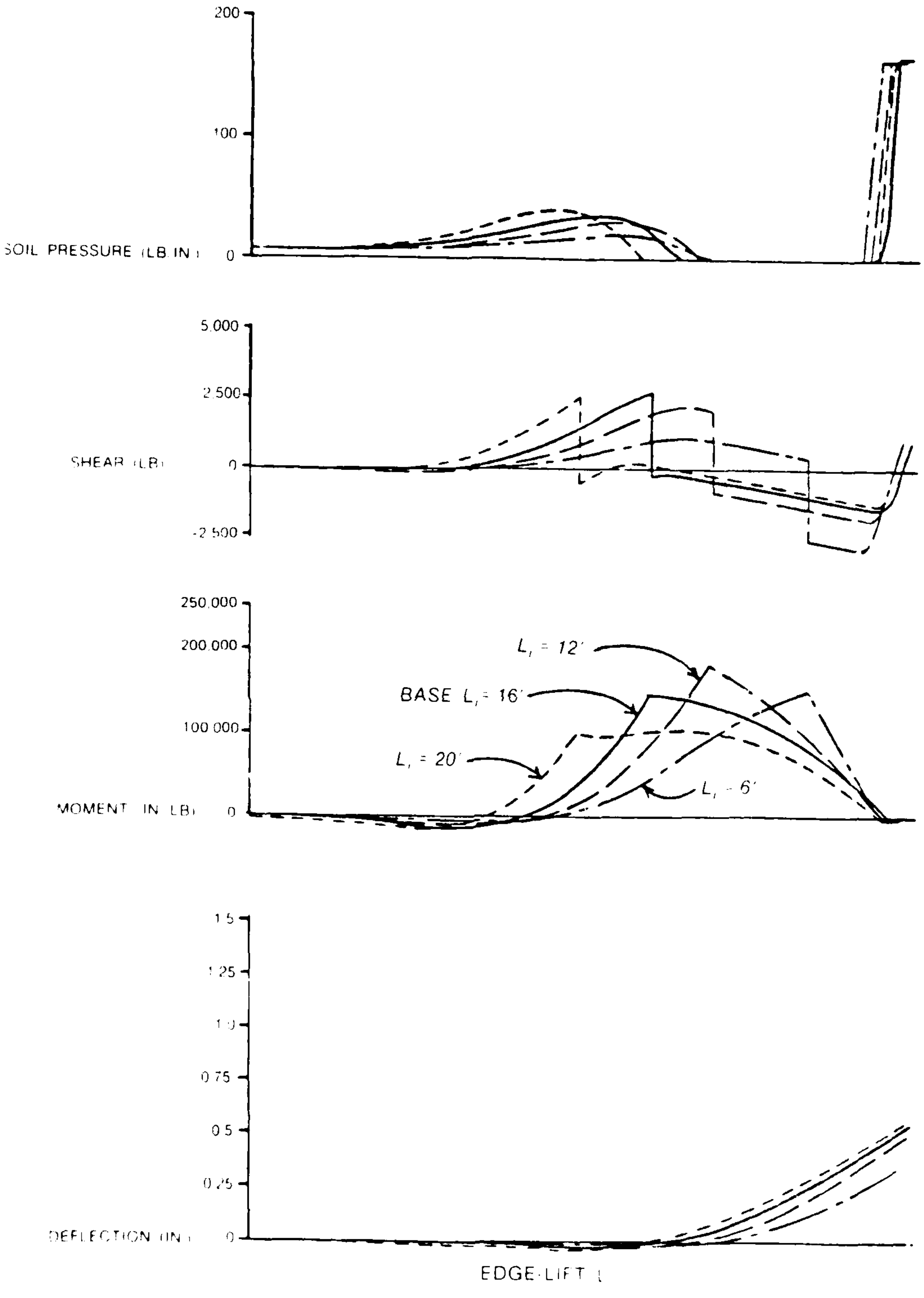

200
100
SOIL PRESSURE (LB IN)
0
5 000
2 500
SHEAR (LB)
0
-2 500
250,000
200,000
S = 16 FT
S = 12 FT
100,000
BASE S 20 FT
S 24 FT
MOMENT (IN LB)
0
1.25
DEFLECTION (IN)
EDGE-LIFT S


200
100
SOIL PRESSURE (LB IN)
0
5 000
2,500
SHEAR (LB)
0
2,500
250,000
200,000
100,000
MOMENT (IN LB)
0
1.5
1.25
1.0
0.75
0.5
0.25
DEFLECTION (IN)
0
P_p 3,000 LB FT
P_p = 0 LB FT
BASE P_p 1,000 LB FT

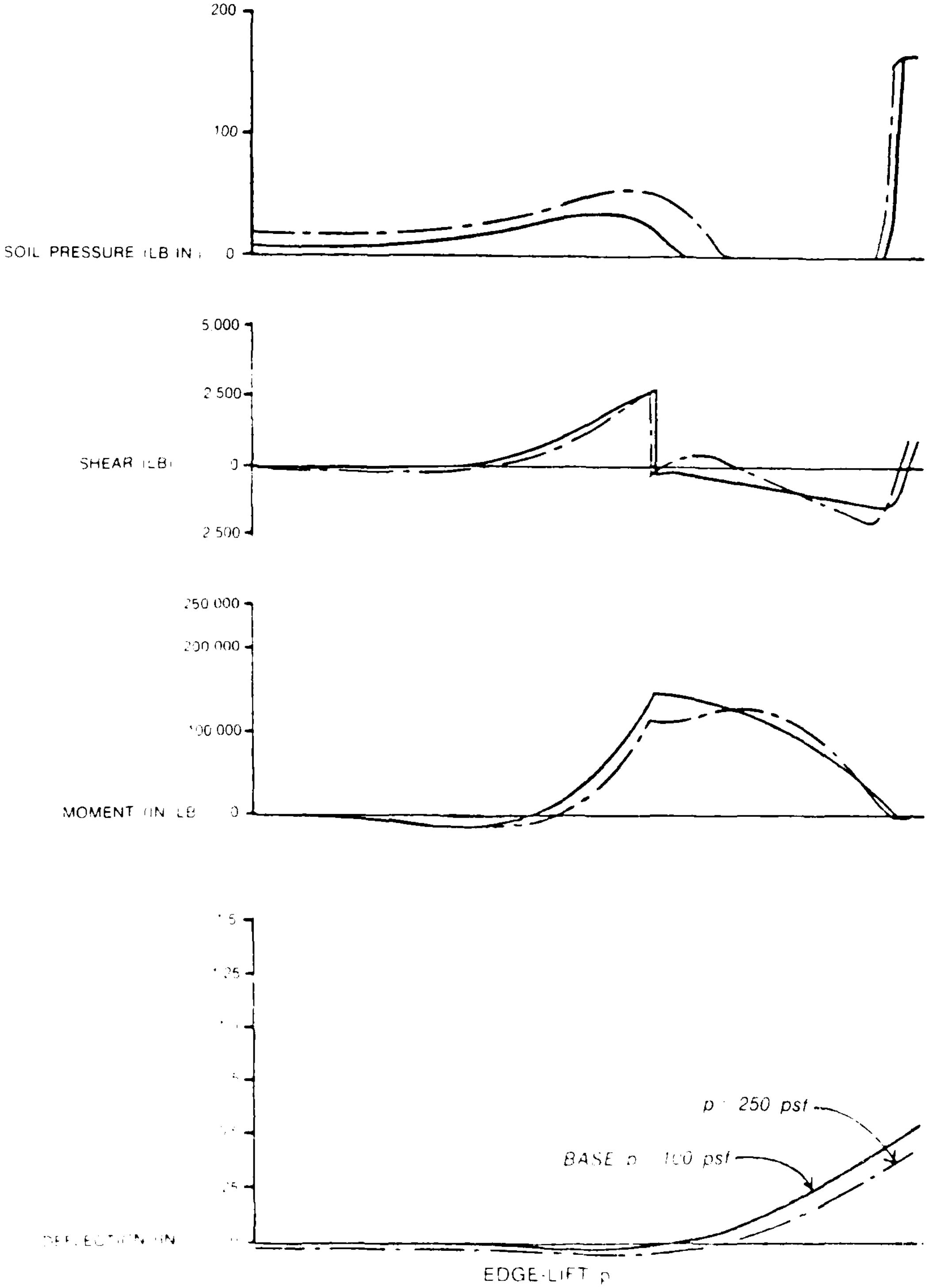
EDGE-LIFT P_L



200
100
P = 5,000 LB FT
BASE P = 3,000 LB FT
P = 0 LB FT
SOIL PRESSURE (LB IN)
0
5 000
2 500
SHEAR (LB)
0
2 500
250 000
200,000
100 000
MOMENT (IN LB)
0
0.5
DEFLECTION (IN)



200
100
SOIL PRESSURE (LB/IN.)
0
5,000
2,500
SHEAR (LB)
0
-2,500
250,000
200,000
100,000
L_i = 12'
BASE L_i = 16'
L_i = 20'
L_i = 6'
MOMENT (IN-LB)
0
1.5
1.25
1.0
0.75
0.5
0.25
DEFLECTION (IN.)
0
EDGE-LIFT


200
100
SOIL PRESSURE (LB IN)
0
5 000
2 500
SHEAR (LB)
0
2 500
250 000
200 000
100 000
MOMENT (IN LB)
0
p = 250 psf
BASE p = 100 psf
EDGE-LIFT p

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	Title	Date
Technical Report K-78-1	List of Computer Programs for Computer-Aided Structural Engineering	Feb 1978
Instruction Report O-79-2	User's Guide: Computer Program with Interactive Graphics for Analysis of Plane Frame Structures (CFRAME)	Mar 1979
Technical Report K-80-1	Survey of Bridge-Oriented Design Software	Jan 1980
Technical Report K-80-2	Evaluation of Computer Programs for the Design/Analysis of Highway and Railway Bridges	Jan 1980
Instruction Report K-80-1	User's Guide: Computer Program for Design/Review of Curvilinear Conduits/Culverts (CURCON)	Feb 1980
Instruction Report K-80-3	A Three-Dimensional Finite Element Data Edit Program	Mar 1980
Instruction Report K-80-4	A Three-Dimensional Stability Analysis/Design Program (3DSAD)	
	Report 1: General Geometry Module	Jun 1980
	Report 3: General Analysis Module (CGAM)	Jun 1982
	Report 4: Special-Purpose Modules for Dams (CDAMS)	Aug 1983
Instruction Report K-80-6	Basic User's Guide: Computer Program for Design and Analysis of Inverted-T Retaining Walls and Floodwalls (TWDA)	Dec 1980
Instruction Report K-80-7	User's Reference Manual: Computer Program for Design and Analysis of Inverted-T Retaining Walls and Floodwalls (TWDA)	Dec 1980
Technical Report K-80-4	Documentation of Finite Element Analyses	
	Report 1: Longview Outlet Works Conduit	Dec 1980
	Report 2: Anchored Wall Monolith, Bay Springs Lock	Dec 1980
Technical Report K-80-5	Basic Pile Group Behavior	Dec 1980
Instruction Report K-81-2	User's Guide: Computer Program for Design and Analysis of Sheet Pile Walls by Classical Methods (CSHTWAL)	
	Report 1: Computational Processes	Feb 1981
	Report 2: Interactive Graphics Options	Mar 1981
Instruction Report K-81-[illegible]	Validation Report: Computer Program for Design and Analysis of Inverted-T Retaining Walls and Floodwalls (TWDA)	Feb 1981
Instruction Report K-81-4	User's Guide: Computer Program for Design and Analysis of Cast-in-Place Tunnel Linings (NEWTUN)	Mar 1981
Instruction Report K-81-6	User's Guide: Computer Program for Optimum Nonlinear Dynamic Design of Reinforced Concrete Slabs Under Blast Loading (CBARCS)	Mar 1981
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Instruction Report K-81-[illegible]	User's Guide: Computer Program for Three-Dimensional Analysis of Building Systems (CTABS80)	Aug 1981
Technical Report K-81-[illegible]	Theoretical Basis for CTABS80: A Computer Program for Three-Dimensional Analysis of Building Systems	Sep 1981
Instruction Report K-[illegible]	User's Guide: Computer Program for Analysis of Beam-Column Structures with Nonlinear Supports (CBEAMC)	[illegible] 1982
Instruction Report K-[illegible]	User's Guide: Computer Program for Bearing Capacity Analysis of Shallow Foundations (CBEAR)	[illegible] 1982

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